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Journal of the
HYDRAULICS DIVISION
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AN ENGINEERING APPRAISAL OF HYDROLOGIC DATA

Progress Report of the Task Group of
Hydrologic Data of the Committee on
Hydrology of the Hydraulics Division

SYNOPSIS

A brief chronology is given of studies made since 1936 on the adequacy of ydrologic data. The resulting reports by governmental, non-governmental ıd engineering groups reveal a continuing need for more and better hydro-gic data for most purposes. The Task Group study shows progress in everal areas, including trends toward faster and more useful reporting data, etter emphasis on analysis and interpretation, use of new methods of data ocessing and need to accommodate new types of data. Recommendations int to strengthening data programs at both state and federal levels. The sponsibilities of civil engineers are outlined.

Historical Background

The Task Group was established by the Committee on Hydrology in late 55 for the purpose of obtaining an objective appraisal, from an engineering ewpoint, of the adequacy of hydrologic data and the current programs of llection with respect to present and anticipated needs. The Task Group ld one meeting, in October 1957, at which preliminary drafts of the report re reviewed and an outline of the final report was approved.

Several studies have been made in recent years in which the adequacy of e nation's hydrologic information has been examined. With two possible ceptions (1949 report of Federal Interagency River Basin Committee and 51 report of Engineers Joint Council), most of these studies treated hydro-gic data only incidentally, or as part of other and perhaps larger problems. ny were considering only the establishment of a suitable water resources licy or the reorganization of certain federal functions.

ote: Discussion open until December 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2079 is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. HY 7, July, 1959.

Some of the earlier studies yielded results and conclusions which are still useful and valid. Others pointed out certain deficiencies, many of which have been overcome in recent years. All problems of obtaining adequate hydrologic data have not, of course, been solved.

Contents

This report should be useful to the Society member and to the civil engineering profession in general because:

1. It is a current engineering appraisal of hydrologic data in the United States.
2. It contains a key to the sources of available hydrologic data.
3. It reveals recent improvements in federal data collection programs and foretells further improvements which may result from current reappraisals.
4. It points out opportunities for making better use of available data and for adding to the volume of public records.
5. It reiterates the civil engineer's responsibility for giving professional direction to hydrologic data programs.

INTRODUCTION

Hydrologic data are used in nearly all walks of life and for many varied purposes. The greatest scientific use, however, is by the civil engineer. The importance of hydrologic data to him is indicated by a definition of civil engineering taken from the California statute regarding registration of engineers; which states, in part:

> Civil engineering embraces the investigation of the laws, phenomena and forces of nature in connection with fixed works for irrigation, drainage, water, power, water supply, flood control, inland waterways, harbors, purification of water . . .

It is obvious that the requirements for hydrologic data and the responsibility for its collection and analysis are a problem of sufficient importance to merit close attention by the civil engineering profession.

Status of Hydrologic Data

Definition of Hydrologic Data

From the point of view of the practicing civil engineer, hydrology is the science of the occurrence, movement, and control of water. The engineer is concerned with the recognition, observation, investigation, analysis, interpretation, and application of all facets of physical hydrophenomena involved in the hydrologic cycle.

Hydrologic data consist, therefore, of the knowledge gained through observation and study of the phenomena in the hydrologic cycle. These become the statistical tools of the hydrologist and the engineer.

In this report, hydrologic data have been classified into three categories: basic data, analyzed data, and interpretive data.

1. Basic data, generally speaking, are observed values or measurements. lso in this category, however, and somewhat distinct from the next group, re observed data which have been converted or otherwise put into usable rm, such as an average or median.

2. Analyzed data include basic data which have been subjected to elemenry degrees of engineering treatment within the limits of the original obserations. Examples are the isohyetal map and the frequency curve.

3. Interpretive data include those items of information which are the reult of interpretation and extrapolation beyond the limits of observation, rough application of sound engineering procedures. An example is the estiated yield of an aquifer. Such data may have many applications to engineerg problems.

Basic data, however, usually are much more than mere statistics. Some xperience and skill goes into even the most rudimental observations. Mere tatistical records often are valueless and possibly misleading unless knowldge of outside influences has been applied. Thus, most observational data ust pass through some analysis and interpretation before being applied to ngineering problems. The following are considered to be the principal types f basic hydrologic data:

precipitation	ground-water levels
evaporation	temperature
snow water content	chemical quality
stream flow	sediment load of streams
river and lake levels	sedimentation in reservoirs

Closely related to hydrologic data are other observations and technical owledge in the related fields of meteorology, agronomy, geology, and other iences. Such factors are useful to engineers in applying hydrology to their oblems.

resent Status of Hydrologic Data

The status of hydrologic data is continually changing with the severity and ope of water problems and with subsequent modifications in programs of ater investigation. Thousands of items of hydrologic data are being oberved, collected, and developed each day throughout the entire field of drology. It is estimated, for example, that more than three million items basic hydrologic data are collected each year in California alone. The thering and processing of an astronomical number of items is part of the gineer's responsibility of giving guidance to data programs and of applying und engineering principles to the use of data.

In fulfilling his duties as a director of hydrologic data programs, the enneer may be required to broaden his vision to encompass both a rapidly owing science and a new concept of its importance. While the collection data at present is generally for a specific purpose, we cannot today acrately predict the use or purpose to which today's data may be applied in e future.

The present status of hydrologic data is a remarkable achievement, condering that the United States was already in its third century of history en systematic public collection of river stage and rainfall records began. elatively speaking, however, most of the records are short-term. The

networks of gages for measuring stream flow and precipitation now constitute both a systematic and a long-range approach to the collection of hydrologic data. With the growing awareness that data needs are constantly changing, the networks themselves should receive periodic reappraisals.

Research also contributes to the knowledge of the physical processes involved in the water cycle. This knowledge helps to determine what features should be measured in the basic data programs. The entire field, therefore, is under surveillance as never before in the short history of hydrology.[1]

Present Status of Collection Programs

Hydrologic data are collected by Federal, state, interstate, regional and local agencies of government, and by private firms and individuals. The Federal Inter-Agency Committee on Water Resources has compiled a complete record of the federal programs in a report which has had only limited distribution. The following summary is based largely on that report.[2]

Federal Program

Department of Agriculture

Within the Department of Agriculture, basic hydrologic data are collected in the Agricultural Research Service, the Soil Conservation Service, and the Forest Service. The data are needed to carry on the authorized functions of the Department.

The Agricultural Research Service collects basic data on precipitation, runoff, infiltration, ground water, erosion, sediment yield and reservoir silting, soil moisture, evaporation, and consumptive use, and makes interpretations of these data for solution of agricultural problems.

The Soil Conservation Service collects basic data on irrigated lands and, i portions of the West, sponsors and coordinates snow surveys. It also collect some data on consumptive use, floods, and infiltration for use in project design. The SCS sometimes allocates funds to the Geological Survey and the Weather Bureau.

The Forest Service collects basic data on precipitation, interception, infiltration, runoff, humidity, and consumptive use at experimental forests and ranges. It collects data on snow, floods, and reservoir sedimentation in forested areas for use in developing good forestry practices.

Department of Defense

The Corps of Engineers, U. S. Army collects hydrologic data for designin and operating its flood control projects and navigation facilities. In addition, it allocates over a million dollars annually to the Weather Bureau, the Geolo cal Survey, and other agencies for the collection of basic data on precipitatio streamflow, sediment load, and ground water. The Corps collect basic data on precipitation and streamflow at sites where the data are needed in curren operation; data on wind, tides, and evaporation for use in design; stage data at locks and dams; and sediment-load records on streams flowing into its reservoirs. As part of its work for the Mississippi River Commission, the Corps collects and publishes streamflow data on the Mississippi River and some of its major tributaries. The Lake Survey office of the Corps collects basic data on hydrologic factors affecting the level of the Great Lakes.

epartment of Commerce

Within the Department of Commerce, the Coast and Geodetic Survey and ne Weather Bureau are authorized to collect and publish hydrologic data for thers as well as primarily for their own use.

The Coast and Geodetic Survey collects basic data on tides to predict their me and height in the tidal reaches of major rivers.

The Weather Bureau is one of the major collectors of basic hydrologic data. is the principal source of basic data on precipitation, evaporation, tempera-ıre, wind, and humidity. Although some of these data may be classified as ydrometeorological, they are all used widely in hydrologic analysis. The /eather Bureau reports weekly and monthly on storms, current floods, roughts, and soil-moisture conditions, particularly as these affect agricul-ıre. It has responsibility for forecasting flood stages and for making water-upply forecasts in the West and in the upper Missouri River basin.

epartment of Health, Education and Welfare

The Public Health Service is the principal Federal source of data on water ollution. It reports on waste disposal, municipal use of water, biological, acteriological, and chemical quality of surface and ground water when as-ociated with pollution abatement.

epartment of Interior

Within the Department of Interior, basic hydrologic data are collected by ie Fish and Wildlife Service and the Bureau of Reclamation for designing and erating their projects, and by the Geological Survey.

The Fish and Wildlife Service measures the temperature of surface water r use in connection with fish-hatchery operations.

The Bureau of Reclamation allots money to basic data agencies such as the 'eather Bureau and Geological Survey for collecting data on precipitation, aporation, streamflow, sediment load, and ground water for existing or pro-osed reclamation projects. The Bureau of Reclamation collects data on con-ımptive use, storm precipitation, and historic floods for designing and erating irrigation and power projects in the 17 western states. It also col-cts discharge records on some of its canals; precipitation records at its ojects; and, in cooperation with other agencies, basic data on density cur-ent, reservoir evaporation and sedimentation, and snow melt.

The Geological Survey, another basic data agency, collects, analyzes, and ıblishes many types of hydrologic data for others to use in developing water ıpplies, abating pollution, and protecting against floods. It is the principal urce of basic data such as stream flow, sediment load, chemical quality, d ground water. It reports on floods and droughts as they affect streamflow d ground water. It collects data on water temperature, consumptive use, d reservoir sedimentation, and collaborates with other agencies in collect-g data on density currents, evaporation, snow surveys, and tides.

dependent Agencies

The Tennessee Valley Authority collects hydrologic data in connection with e operation of reservoirs and area development. It collects and publishes ta on: reservoir stages, evaporation, density currents and sedimentation; filtration, interception, soil temperature, soil moisture and ground water at

experimental areas; and chemical quality and water temperature of reservoirs and streams. Much of the stream gaging in the TVA area is financed by transfer of funds to the Geological Survey.

The International Boundary and Water Commission collects hydrologic data in connection with carrying out the provisions of the Mexican treaty for appropriation of the water of the Rio Grande. It measures streamflow, floods, sediment load, chemical quality, and reservoir storage on the Rio Grande and tributary streams between Elephant Butte Dam, New Mexico, and the Gulf of Mexico. In addition, it collects some data on precipitation, air temperature, humidity, and wind in that area. In the Lower Colorado River basin it collects streamflow records at several sites.

State and Interstate Agencies

Departments of State Government

State geological surveys, conservation commissions, water engineers and other agencies of state government either cooperate with federal agencies in collecting basic hydrologic data or conduct independent programs. In some states the cooperating parties share both in the financing and in the engineering work, while in others the cooperation is entirely financial. Most of the states having cooperative programs with federal agencies rely on the federal publications to release the statistical data and confine their publications to summaries or interpretive reports. Some state agencies collect data which is of state interest only, such as might be obtained in a census. States often obtain data through research and experimentation at state universities.

Additional data of state interest are gathered by state organizations having legal or administrative responsibilities in conjunction with water problems. This is especially true of states west of the hundredth meridian that administer an appropriation doctrine.

Interstate Compact Commissions

Interstate agencies exist for a number of major streams in the United States. Generally speaking, the collection of data is done through the component state organizations.

Regional and Local Agencies

Regional and local agencies include authorities, cities, towns, counties, special districts, etc. Some of these agencies collect basic hydrologic data required for the fundamental purpose of the organization. For example, sanitary districts collect data on stream height and flow, waste water discharge, water quality, and weather. Such local data may be exceedingly valuable to persons carrying out regional hydrologic studies, but it usually is on file only at each local institution.

Water supply authorities, districts, and utilities serving municipalities assemble data on ground water and surface water sources being used. They seldom assemble data on sources not being used, and in many instances data are lacking even on the sources being used.

Private Firms and Individuals

Airlines, forest industries, agriculturalists, utilities and other private erests may collect and maintain records of hydrologic data for their own e. In nearly all cases the collection is related to the specific business tivity and is therefore specialized both as to type and location.

Engineers and consultants in private practice are not ordinarily collectors basic data for public use. Some engineers, however, make their records ailable to collecting agencies, but few are published. Private records and ta from public sources are used by engineers and consultants to develop w or related data. Such data usually become the property of the client and e never published. Some may appear in technical articles in national rnals.

Policies Governing Data Collection

Flowing water differs from land and other fixed property in the important spect that it cannot be retained for use by one person but passes on to other ers, thereby creating complex problems in water rights and damages. This ndition makes necessary the collection by government of important basic ormation for public use.

The Geological Survey and the Weather Bureau are the two agencies having mary federal responsibility to collect and publish hydrologic data. The icies which govern their operations have been described in statements omitted by their chief executives to the Task Group. The complete state-nts are included in the Appendix.

sic Authority

The Geological Survey was created in the Department of the Interior in 79, but its responsibilities for water resources investigations were not de-ed until several years later. The first appropriation was made in 1888, l successive appropriation acts have continued to define its authority to uge the streams and determine the water supply of the United States, in-ding the investigation of under-ground currents and artesian wells in arid l semi-arid sections." Its cooperative investigations with the states have en a major part of the work for more than 50 years, and emphasis on this operation is increasing.

Objectives of the Geological Survey's programs are the determination and scription of the quantity and quality of the surface and underground waters the country — an appraisal of the total water resources. Over the years emphasis has shifted from mere collection of basic data to the analytical l interpretive phases of water resources investigations. Research and ex-imentation are becoming important adjuncts of the Survey's investigative ponsibilities. The ultimate objective is an improved knowledge of natural nomena as the basis for planning and design of water resources projects.

The Weather Bureau was organized under the Department of Agriculture 1890 and was transferred to the Department of Commerce in 1940. In re-ding and determining the general climate of the United States and its pos-sions, the Weather Bureau maintains weather, climatological, and hydro-ic services. A central repository for its data is provided at the National ather Records Center, Asheville, North Carolina. Although the Weather

Bureau cooperates with other federal agencies, its cooperative investigation are limited by lack of authority to accept non-federal funds.

The Weather Bureau has responsibility for forecasting weather and river conditions, and for summarizing, analyzing, and interpreting related hydrolo ic data for general use. It maintains a climatological and hydrologic networ of observation stations throughout the country.

Program Reappraisals

The Geological Survey is conducting a comprehensive reappraisal of the effectiveness of its data collection programs, leading to possible adjustment and improvements geared to present and future needs for hydrologic data. The results of network reappraisals will be published, probably in summary form, as soon as practicable.

Revisions are under way in the stream gaging network, which is compose of three principal elements: a primary network of long-term stations, a secondary network of relatively short-term stations, and a third category of project operation or management stations. Increases are proposed in both primary and secondary stations to a total of about 10,000 active gaging stations in the network.

Study continues on the stream sediment station network and the daily chemical quality network. These analyses are expected to provide objective and systematic approaches to the needs and improved criteria for establishing the stations.

The Weather Bureau has completed its network reappraisals and has beg to modify its station objectives and locations. Two types of stations are included in the climatological network: a few long-term "bench-mark" station selected for permanence, and a greater number of short-term stations. Ma observation stations are incorporated in both the climatological and the hyd logic networks. The revised hydrologic network will be based on known nee of particular projects and investigations rather than on area distribution. T stations are less permanent and can be shifted to meet changing conditions. Both networks are shown on maps available at offices of the State Climatologists.

Factors Affecting Magnitude

The size and scope of programs for collecting hydrologic data are influenced by many factors which vary with changing economic conditions. Amon the factors which may exert an influence on the magnitude of data collecting are: availability of funds, usually public; population growth and increasing demand for more water, for industrial, domestic, and agricultural purposes availability of trained manpower to serve the collecting agencies; problems of stream pollution resulting from urbanization and industrial activity; and such new developments as weather modification and disposal of radioactive wastes.

Financial limitations

One of the most important factors in maintaining an adequate data base i the availability of sufficient funds. In general, more and better data could b collected with additional money. Legislators and administrators, however,

both federal and state levels, frequently have not provided sufficient ap-
opriations for carrying forward a stable, comprehensive program of data
llection, processing, and publication. The results of operating with insuf-
ient funds are inadequate areal coverage, discontinuous records, and de-
iencies in quality of data. As measured in terms of what existing technical
lities can provide, however, most of the publicly financed programs are
lding nearly maximum possible results with the funds available.

reasing Uses

The accelerating use of water by municipalities and industries has been
enomenal. In 1924 the United States had about 9,800 domestic water supply
stems, and in 1950 the number was close to 17,000.(3) Industrial growth is
en greater, and further expansion is predicted; industrial water use of 80
lion gallons a day in 1950 is expected to be 215 billion by 1975. Both public
l private enterprise in the water works industry need more data for planning
v supplies and more efficient utilization of available water resources.
Agricultural use of water increases as more land is brought under irriga-
n and as supplemental water is applied to growing crops in humid areas.
e 1954 U. S. Census of Agriculture reports an increase of more than 30
rcent in the country's irrigated acreage between 1944 and 1954. Water use
l control must be based on adequate hydrologic and agricultural data.
Additional hydrologic data are needed regarding water quality in most
eams in order to continue to solve the pollution abatement problem. The
eral government has a responsibility under the Water Pollution Control
t of 1956, while the states have a primary responsibility for controlling
lution within their borders. Both federal and state agencies depend on
owledge of river hydrology to measure the results of abatement programs
l to set standards for treatment of effluents.

npower

Even though adequate funds may have been provided, the trained man-
ver to carry out the necessary investigations has not always been available.
s was particularly true during the war years, but it has continued beyond
recent war. Adequate staffing of the government scientific agencies,
ch are under Civil Service, has been hampered by salaries not competitive
h industry. Some segments of the engineering profession are advocating a
ecial civil service rating in these categories as an aid to improving the
npower situation.

er Factors

The increasing use of various forms of weather modification techniques
icates need for accurate and unbiased verification of results. The fact that
orted results of "rainmaking" often conflict with each other is an indica-
n that special data must be obtained in areas where weather modification
being attempted. Adequate information collected and analyzed in accordance
h engineering standards, is the only reliable measure of the effectiveness
current weather modification techniques.
In anticipation of the time when streams will be carrying new pollutants
other substances not now present, early effort should be made to initiate
a collection programs which will establish a base period of record.

Water management problems are increasing with expanding use of water, and the more critical the problem becomes the greater is the need for accurate and extensive water data. Growing competition for limited water sup plies increases the demand for better data as a basis for equitable allocatio

Processing, Analysis, and Interpretation

Treatment of Basic Data

It is often difficult to separate the collection of hydrologic data from the processing, analysis, and interpretation. Observed data, for instance, may be of little value to the user unless made part of a more comprehensive stu in which basic data have been related to other facts. The collector of basic data has a responsibility to analyze and interpret to a degree that the end product is useful in several ways to others.

The primary functions of most federal agencies which collect hydrologic data are not the collection of data but the operations which require the use application of data. Even the Weather Bureau and the Geological Survey, th most likely exceptions, were established for other purposes. Nevertheless, data are collected, interpreted, and published as an authorized function of t federal government in promoting the general welfare. It follows that some degree of data treatment also is a proper federal function.

Some processing of basic data is necessary in order to put it into forms usable by others. The broad general investigation and its direct results in terms of data are the primary responsibilities of the collecting agency. Th adaptation of the data for specific uses is a responsibility of the user.

It may be impracticable to define the extent to which each type of data should be processed at public expense. It is obvious that, in general, meag processing is undesirable and excessive processing is unwarranted. The ex perience and background of the collector, which may be useful to the user, are lost if only "raw" data are presented. In contrast, too much processin may infringe upon private engineering, waste public funds, and thus advers affect the collection of the basic data.

New Methods and Equipment

Advances in the last decade have been greater in the field of data proces ing than in the field of data collection. Standard methods and equipment fo measuring various hydrologic factors were largely developed prior to 1940 New publication devices and media, however, are now making large volume of hydrologic data more readily available. Electronic equipment for rapid scanning, sorting, and computing has greatly improved many of the data treatment processes which must precede publication of records.

Both the Geological Survey and the Weather Bureau are using high-spee electronic computing equipment for analysis of hydrologic data. Water con trol and power system operations are electrically computed by Bonneville Power Administration. These are examples of new developments which ar not only saving time but also are:

- Making more use of existing data
- Opening up new uses of data and permitting new applications
- Reducing the cost of data processing

tralization of Data Programs

Centralization of hydrologic data programs has both advantages and disad-tages. An advantage to the collecting agency might be a disadvantage to data user, and vice versa. Although conditions may vary causing excep-s, the following are some of the advantages and disadvantages:

antages of centralization

Records may be more uniform, consistent, and reliable; costs may be er because of less duplication and higher efficiency; publication and dis-ution may be hastened. Certain data of general interest and importance well adapted to centralization; for example, the Weather Records Center, ville, North Carolina.

advantages of centralization

Early availability with a low degree of accuracy may be better than later lability and higher accuracy; certain data of local interest are not adapta-to centralization; centralization can lead to undesirable degrees of eaucracy and concentration of authority.

Adequacy of Data Collection Programs

The adequacy of hydrologic data is an important consideration in policies sed to use the nation's water resources. Appraisals of water policy, made e 1950 by many official groups, mention the great need for adequate basic . In policy statements on conservation of natural resources adopted by onal organizations of agriculture, business, and industry, numerous refer-es are made to the adequacy of data. These studies and reviews by groups ide the civil engineering profession indicate that the optimum amount of c data is not always available. They recommend financial support to im-ve the basic data programs.

Iydrologic data being collected today, to be adequate, must include not only which is required at the moment but also all that which can be justified anticipated future needs. Historical periods of record give the data real e. The uncertainty of tomorrow's needs makes today's adequacy a matter pinion rather than a certainty. However, the experience of the past gives onable guidance for the future.

Before reaching a decision as to the adequacy of certain data, its relation- to engineering need should be examined. What standards and require-ts are to be met? Is adequacy a relative term; is it changing, becoming e critical? The practical must be weighed against the ideal, realizing mere volume of data is not the real objective.

critical examination of hydrologic data programs shows some over-hasis on volume of data. The need of the user is not always for more but rather for the right data which can be interpreted properly. Those erned with this problem should recognize the danger of demanding more as a substitute for professional ability to use properly what is available.

Nevertheless, most authorities agree that both the quality of data and the uacy of its collection are improving. Furthermore, attention should be n to keeping pace with new needs and dropping whatever is no longer ssary.

The adequacy of six principal types of hydrologic data is viewed as follows e Task Group.

Precipitation

In general, the volume and coverage of basic data are adequate, and the information is readily available. Public and private records of precipitatior throughout the country are well distributed and some are of rather long dura tion. The network of gages for collecting short-period rainfall intensity ma be adequate. The greatest deficiencies are in the mountainous areas, in sm watersheds, and where unusual climatic and orographic conditions exist.

The rainfall intensity-duration-frequency reported in the Weather Burea technical papers has filled a long-standing need. Progress also is being ma in the study of areal distribution of precipitation from local intense storms. Further interpretation of hydrometeorologic data of this type will provide a additional tool for determining runoff characteristics.

Snow-water content observations are increasing gradually and are generally adequate for operational purposes in most of the areas. Modest expa sion of the coverage would be desirable in some areas to build up base records and to increase the knowledge of snowfall and water content.

Other Climatic Data

Except for evaporation data, the coverage is probably adequate from an engineering standpoint. Presently available data on pan evaporation are ade quate except in areas of rugged topography. More basic study of the Lake Hefner type should be carried out to show water losses in relation to other climatic factors, so as to relate pan studies to open-water conditions.

The Weather Bureau reports that a reappraisal of evaporation data on a nationwide basis is under way.

Runoff

With the increased use of surface water supplies, the need for stream fl data has increased and probably will continue to increase over a period of time as the demands for more adequate data expand. However, there will never be justification for measuring every stream at every significant chan in flow. Any future expansion of the network of gages should be based on maximum possible use of correlations, employing data from key gaging stations to estimate (within close limits of error) the runoff at other locations. Additional application of correlation techniques to present records might al low some reduction in the number of gages without jeopardizing accuracy. The most obvious deficiency is in data on runoff from small drainage areas of 50 square miles or less. More data are needed on stream flow in the zo of tidal influence, and data already collected should be assembled, analyzed and published.

The USGS stream gaging network is now undergoing reappraisal to determine its adequacy and the most suitable density to meet desired needs and standards. This reappraisal is taking into consideration the views and nee of other agencies and data users. Some expansion is taking place in analys of runoff records which leads to or assists in later multiple use of data.

Ground Water

Deficiencies in data on ground water are greater than for other types of data. Ground water investigations have been expanded in recent years, how ever, and some of the deficiencies are disappearing.

Attention to ground water problems on an aquifer basis rather than by unties or other political unit would improve the data now being obtained. ploratory work is needed to implement the discovery of new potential urces of ground water. As water uses approach or exceed safe yields, idies of limiting factors will become more detailed and data will become ore important. Publication of information on the hydraulics of aquifers ould be encouraged.

Although a network of federal observation wells is in operation, the extent its coverage is relatively small. Federal participation in data collecting s been promoted primarily in cooperation with state and local agencies. der the cooperative programs, areal coverage is increasing, but much of data pertains to local problem areas and thus only indirectly increases basic knowledge of ground water resources.

Some ground water data are collected by private investigators but little pears in print. Only a few states require public filing of well records, ter levels, logs, and pumping data.

ality of Water

Inadequate information on chemical quality and temperature of water is re often due to lack of uniform, readily available publication than to a lack investigation. Better coordination and standardization of field practices l reporting of data are needed. Water temperature should receive more ention in areas where it is becoming increasingly important for industrial e.

The USGS is appraising its sampling network for chemical quality data in eams. The results may indicate the deficiencies and adjustments needed the network.

limentation

The present sediment sampling program is somewhat less than comprensive. On the other hand, intensive sampling may be unnecessary after ationships between stream flow and sediment have been established. Some npling probably can be abandoned entirely. The study of the program now ler way by the USGS should show where readjustments can be made to give ter results with no increase in costs.

Data on the accumulation of sediment in reservoirs are generally adequate l should be continued. Small reservoirs are not as well investigated as ge ones, however. No one federal agency needs to be given primary responsibility for this work if there is adequate coordination among agencies, h as through the Federal Interagency Committee.

Publication of Hydrologic Data

blished Reports

Of all the basic hydrologic data collected in the United States, only a porn is prepared for publication in reports. The regular publication media of Weather Bureau and Geological Survey are more complete than they were years ago. In general, they are meeting the needs of the engineering proms - especially those involving large river basins and major drainage as. The Federal Interagency Committee on Water Resources has compiled

useful inventories of data on water quality and sediment load.

In recent years the demand has increased for hydrologic data in the smal ler streams. Here the regularly published data lack the detail necessary to develop sound engineering plans for watershed protection and flood preventi in small areas and for highway culvert design. A few special reports have been issued which help to meet some of the demand for this type of data.

Many private collectors of hydrologic data have no plans for publication o their basic data. There is no established procedure to have such data included in regular or special publications of federal or state agencies. Data thus collected for a special purpose may not be suited for publication in jour nals of scientific societies or in special bulletins of private or public organi zations. In such form, also, the basic data might not be sufficiently complet to afford a check on conclusions which may be drawn. Much unpublished dat is filed upon completion of the study, and is either lost or destroyed.

The time lag in the publication of precipitation data has been greatly reduced in the past few years, now being generally less than two months for hourly and daily precipitation. Stream flow data appears in print about 24 months after the end of the water year, the delay being due to the time required for analysis and review and to a backlog of computation work in some field offices. New procedures being considered for stream flow records promise to shorten the lag time for publication by several months. Decreas ing the present lag time for precipitation data does not appear justified.

Descriptions of the regular and special publications of all the federal agencies dealing with hydrologic data are given in the previously noted repo "Principal Federal Sources of Hydrologic Data."

The periodic and serial publications of most agencies are improving, but the need seems to be for different types of reports. There is need for more special summaries, such as those issued by the Geological Survey and the Weather Bureau. Some should be issued more frequently.

Unpublished Records

It has been recommended that some means be provided for publishing dat collected by those who have no publications program. Examples are the vas amounts of sediment data collected by the Corps of Engineers and the information on water quality collected by numerous agencies. The volume of unpublished hydrologic data constitutes one of the profession's valuable resources. How to make it more readily available and useful is a challengi problem. The designation of selected agencies as official repositories for specific data has been proposed.

The public agencies now devote large portions of their money and manpower to preparing data in forms most useful to practicing engineers. They constantly seek to produce better and more useful publications. If what they prepare is inadequate, the fault may be only partly theirs. If the need is fo different forms of data, the prospective users have an obligation to see that the public agencies are fully informed.

New Methods and Analytical Techniques

The new high-speed methods of machine processing are of proven value reducing the time lag in publication of basic data. For such purposes, it is necessary for the data collection agency to place the data on machine punch cards or tapes. With the data available in this form, new fields have been

pened for ready processing of analyzed or interpretive data by the collection gency. The cards or tapes can be used and reused for this purpose.

Advantage may well be taken of high-speed machine methods to increase he amount of analyzed or interpretive data to be issued by the collection gencies. The limit of such analysis and interpretation is difficult to establish. However, the data collection agencies should not unduly emphasize the nalysis and interpretation at the expense of basic data collection. The following guide lines are considered applicable:

1) Collection of basic data should take priority over analysis and interpretation. Analysis and interpretation can take place at any time, but in most cases hydrologic data can only be collected at the time of the physical occurrence. Recognizing that a practical limitation will always be put on the amount of funds available, analysis and interpretation should not be overemphasized so that the basic data collection program is allowed to fall below a generally acceptable level.

2) Analyzed or interpretive data should be processed if there is reasonable expectancy of subsequent multiple use, but should not be processed if expected to serve a single purpose.

3) In most cases, the collection agency is best qualified to make interpretations. The basic data used in the interpretive analysis should be available to the user for his own interpretation.

Within these general limitations the collection agencies should produce nore analyzed or interpretive data. Some of the data which can be produced re: rainfall intensity probability, streamflow frequency, summaries of elected occurrences such as maximum streamflow or precipitation, and ummaries of monthly streamflow or precipitation values.

The data available on cards or tapes have also been used in other ways. opies of cards or tapes can be made for the user who can then program, on achines available in his own locality, the analytical processing he requires. uller use can be made of cards and tapes as engineers become aware that asic data are available in this form and are informed of the procedures for btaining it.

The high-speed methods of machine computations have additional implicaons in the data collection program. The use of electronic computers for nalytical and interpretive purposes can be expected to increase, and as such se increases, the ability of the user to consider greater amounts of data ill correspondingly increase. The result could well be a demand for greater mounts of data, whereas in the past such demands may not have been exerted ecause of the impossibility of processing voluminous records by manual ethods.

ecords Storage

The feasibility of storing old and unused records on microfilm has been ell proven. Where records are used often, however, the use of microfilm frequently unsatisfactory. Decision on whether or not to use microfilm ust be made by the data collection agency on the basis of its own knowledge the probable future use for the original data. As one of many factors in e internal efficiency of the data collection agencies, it would appear to have earing on the adequacy of hydrologic data only to the extent that efficiency fects the overall program.

New Types of Data

Most authorities agree that water supplies will become critical in more and more places in the years ahead. All trends of water use point to intensified competition for water and ways to use it. Few engineers 100 years ago appreciated the need for maintaining records of stream flow, but none doubts the value of such records today. Twenty years ago no one thought of testing our water courses for radioactivity or to determine minute amounts of trace elements. Such tests are only now becoming important to engineers and natural scientists.

Knowing well that mankind cannot greatly increase the amount of water received on the earth, most of our hydrologic-engineering problems involve better use of what we have, using it wisely and treating it well. For example, evaporation generally is a waste of valuable water, and if evaporation can be reduced the effective water supply will be increased. Encouraging results have been obtained from experiments with the use of monomolecular films as inhibitors of evaporation from water surfaces.

In other fields as well, such as sewage reclamation, new opportunity must be found to develop good water supplies when and where needed. The reclamation and reuse of water have been practiced for a long time, but more must be learned of the potentialities in the United States. So far reclamation has been developed principally in areas of critical water supplies where it is feasible to process a large volume of sewage for industrial or irrigation use. Hydrologic data are needed on both quantity and quality of polluted waters, including not only domestic sewage but also saline (natural) waters which might be reclaimed. New data may be needed when the demineralization of saline and sea water approaches a point of economic feasibility.

The use of radioactive materials in manufacturing, medicine, and the production of electric energy, as well as nuclear reaction for defense purposes, will increase water pollution unless great care is exercised. The collector of hydrologic data is one of the watchers in this new field. New data should be obtained on rates of contamination and on the natural processes of purification.

Additional comprehensive hydrologic data are needed to determine the effects of watershed treatment measures. Depending on the physiographic, climatic, and other conditions, such measures may have various effects, both hydrologically and economically. More data are required to assess such effects for different parts of the country. It is noted that a joint study to develo procedures for making determinations of hydrologic effects is being carried out by the Agricultural Research Service, Soil Conservation Service, and Bureau of Reclamation.

Artificial nucleation of clouds to increase precipitation has been the object of research and actual operation for more than a decade. A recent report indicates that at one time more than 20 percent of the United States was being subjected to rainmaking efforts. Commercial firms claim considerable success, and beneficiaries willingly pay for their services. In contrast, however the World Meteorological Organization reported in August 1955 that a net increase of precipitation has not been demonstrated beyond reasonable doubt in any seeding operations yet described in technical literature. Yet, it is incorrect to state categorically that rainmaking is impossible. Much orderly research will be required to establish the potentialities of weather modification. Statistical treatment of hydrologic data is the only way in which the results of seeding operations can be assessed quantitatively.

Instead of a frontal assault on water problems by great masses of basic data, long-range planning should be applied to determine new needs and new types of data. It seems obvious, for example, that more complete and accurate data is needed where prospects of water problems are most iminent. The civil engineer should exercise the imaginative thinking necessary to provide a capacity to collect and interpret hydrologic data which will be equal to the sum of all expanding demands.

CONCLUSIONS

The following conclusions have been drawn from the study by this Task Group:

1. The increasing urbanization and industrialization of the United States is requiring continually greater amounts and better quality of hydrologic data. Considerable improvement has taken place in recent years, particularly within some of the federal agencies, but further progress is necessary. In general, adequate hydrologic data are being collected to meet most engineering needs. As deficiencies in basic data are overcome, however, more emphasis must be put on analysis and interpretation in order to increase the availability and usefulness of the data.

2. Deficiencies continue to exist in certain types of data: precipitation and runoff data in small drainage areas apart from agricultural and industrial developments; ground water data in undeveloped and newly developed localities; and water quality data in a number of scattered regions.

3. Publication of many types of hydrologic data is improving and the time lags are decreasing but not in proportion to the growing needs. New kinds of reports, such as periodic summaries, are making some of the widely used data more readily available. New machines and processes are capable of reducing the time (but not necessarily the cost) required to produce usable records of basic data, thus leading to greater productivity by professional personnel both in processing and in using the data.

4. If a suitable means could be devised for processing and publishing miscellaneous data from public and private files, valuable records now lying useless would be made available to the engineering profession.

5. Inadequate federal and state appropriations have held back some worthwhile data collection programs. Progress is hampered by lack of qualified professional personnel who are not attracted by government salaries which are below industrial rates.

6. The existing federal networks of gages measuring hydrologic data are producing satisfactory records. Periodic reappraisals are desirable for bringing about improvements to keep pace with changing needs and conditions. A reluctance to change prevailing practices can lead to overcollection and failure to begin records needed for the future.

7. The cooperative programs between state and federal governments are yielding satisfactory data on streamflow and ground water. The extension of cooperative programs to other types of data might help to retain state responsibilities as well as to reverse trends toward overcentralization in federal agencies.

8. The coordination of federal and state activities in data collection through regional interagency committees is desirable. Similar coordination of federal activities among various agencies should be encouraged.

9. The major responsibility for producing and utilizing the nation's hydrologic data rests with the civil engineer.

10. Equally as important as the availability of adequate hydrologic data is the engineering competence required to interpret and apply the data for beneficial purposes.

Recommendations

The findings and conclusions of the Task Group lead to the following recommendations:

1. Federal-state cooperative programs of basic hydrologic data collection should be extended. In addition to strengthening the cooperative programs for stream flow and ground water, similar programs should be established for other types of hydrologic data, such as hydrometeorological, as recommended by the President's Advisory Committee on Water Resources Policy. Such programs should be encouraged at the state level.

2. A satisfactory federal hydrologic data program can be obtained through increased coordination and cooperation among agencies. Scattered elements of basic data collection should be regrouped in order to reduce the number of agencies involved to a workable minimum, but all programs should not be centralized in a single federal agency.

3. Sound conservation and development of the nation's water resources requires more adequate hydrologic data which can best be derived by applying the technical abilities and the professional integrity of civil engineers and others with similar qualifications. Therefore, competent persons should be in charge of designing hydrologic data programs and supervising the collection, processing, and scientific use of hydrologic data.

4. Having both a professional and a civic responsibility with respect to hydrologic data, the private civil engineer should maintain adequate records so that data no longer needed for exclusive private purposes would be available for publication by a public agency.

In conclusion, the Task Group appreciates the opportunity to serve the Hydrology Committee and the Society. The Chairmen sincerely appreciates the assistance and cooperation of the members of the Task Group, the members of the Hydrology Committee, including former Chairman R. K. Linsley, W. F. Guyton, and H. S. Riesbol, and particularly C. C. McDonald who served as Task Group Coordinator.

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APPENDIX I — STATEMENT BY U. S. WEATHER BUREAU (November 25, 1957)

Reappraisal of networks

A reappraisal of the basic data network, state by state, was begun about four years ago and is now completed. The reappraisal had two purposes. One was to determine the required density and distribution of precipitation and temperature observations for adequate definition of the climate. This determination has been followed by authorization of a representative grid of observation stations, uniformly spaced over areas of little climatic gradient and increasing in density with increase of climatic variation or gradient. Unnecessary stations are now being discontinued, and new stations established in accordance with the plan and available funds.

A limited number (25-50) of stations with long records and excellent prospects of continuity will be designated as bench-mark stations, and the records, after thorough evaluation, will be used to determine secular trends.

The next step in the climatological network (still in the planning stage) is to utilize the double category climatological network plan. Such a network would consist of one group of stations which we expect to keep unchanged almost indefinitely, and another larger group, each of which would be moved every 5 or 10 years to provide data from a greater number of locations. Data from these short-term stations could then be correlated with data from nearby long-term stations to provide an estimate of the long-term climate for the specific local area.

The other purpose of the reappraisal was to establish the known supplemental requirement for observations of evaporation, wind, humidity, soil and water temperature, as well as precipitation and air temperatures, to meet the needs for both analysis and operation in the field of applied hydrology. A large part of this requirement was based on the 1949 recommendations of the Federal Inter-Agency River Basin Committee. This hydrologic network includes portions of the climatic network but does not follow general rules of distribution and density of observations. It is based, rather on the known needs of particular projects and investigations. It is, therefore, flexible and subject to change; and its implementation depends on project authorization and availability of funds for these projects.

The networks are shown on state maps used as working tools in our substation management program. Publication is not intented but they can be inspected at State Climatologist Offices and copies made available if required.

General and specific policies

The Bureau's general policy is fulfillment of its obligation to record and determine the general climate of the United States and its possessions, at the same time making available the basic observational data by archive, punch card, microfilm and publication. In addition, the Bureau has extensive forecasting responsibilities, both for weather and river conditions, for which additional observations are required and these, too, are incorporated in its basic data program. For the more specific studies, both climatological and hydrologic, required by other agencies and specialized interests, and for operational use, the Bureau provides its facilities, its advice, and its cooperation whenever possible. For all these data, and for other data made available to it, the Bureau provides the national repository at its National Weather Records Center at Asheville, North Carolina.

The extent of summary, analysis, and interpretation of these basic data is not spelled out, except that the Bureau's self-supported limited program along those lines is naturally for general rather than project use, unless the project is the improvement of the Bureau's forecast procedures or climatic presentations. However, as in the observational program, the Bureau also provides its facilities, advice, and cooperation to meet other agency or inter-agency needs in summary, analysis, or interpretation. The numerous Hydrometeorological and Cooperative Studies Reports, Weather Bureau Technical Papers Nos. 2, 4, 10, 13, 15-17, 23-26, 28 and 29, and Weather Bureau Research Papers Nos. 34 and 38 are cited as examples of what can be done when funds are available; in a majority of these cases the costs of these studies were substantially supported by transfers from other Federal agencies.

Adequacy of basic legislation

The cooperation of Federal agencies has accomplished much in the advancement of basic data programs, both operational and analytical; and the cooperation has been facilitated by inter-agency transfer of funds. The same facility is not available to the Weather Bureau for cooperation with State or other non-Federal organizations, although the Presidential Advisory Committee on Water Resources and Policy has recommended extension of Federal-State fund-matching and similar authority to all Federal basic data agencies. Certainly extension of this practice to the meteorological and hydro meteorological field would stimulate State interest and permit intensification of networks and special investigations considered to be of insufficient national interest to justify Federal expenditures. The purpose, as in Federal inter-agency cooperation, would be to realize the technical and economic advantages of specialist processing and interpretation of basic data into the form required by the design specialists of the agencies responsible for development and operation of water utilization, conservation and management projects.

Data programs under a single agency

Centralization of basic data programs within a single agency should provid the usual advantages of more uniform standards, closer coordination of related programs, and elimination of duplication. However, coordination and cooperation among the several agencies concerned are now very good.

Witness, for example, the increase in multiple-purpose observational stations serving several agencies; transfer of funds from agencies like the Corps of Engineers, Bureau of Reclamation, and Soil Conservation Service to the Weather Bureau for the establishment of meteorological stations and for special hydrometeorological studies; similar transfers to Geological Survey; and cooperation between Weather Bureau and Geological Survey in preparation of hydrologic atlases and flood reports.

There is thus a natural, healthy growth toward realizing the advantages of single-agency centralization without foregoing the advantages of specialist and professional supervision of diverse aspects of the basic data program. The meteorological and climatological data of the Weather Bureau, for example, are collected for a great variety of uses, of which only some are hydrologic. There is no more reason why they should be collected by a hydrologic data agency than that they should be collected by an agricultural, aeronautic, health, manufacturing, or marketing data agency. They are essentially meteorological data and are likely to be most efficiently and professionally collected and processed by a basically meteorological agency. Even the Bureau's most strictly hydrologic data, such as river stages, are so intimately tied in with meteorological observations and forecasts for river forecasting purposes that they are most logically collected within the same channels and under the same auspices. The single-agency concept, it seems to us, arises from an over-simplification of the problem. A more realistic approach would be to consider a regrouping of elements of the basic data programs of the various agencies, in order to bring together those elements most closely interrelated within the appropriate agency.

Federal cooperation with State and local agencies

This could well be increased and is discussed under another heading. The factor to guard against in such cooperation is the year-by-year uncertainty of financial support that would jeopardize the continuity of records and make planning difficult.

APPENDIX II – STATEMENT BY U. S. GEOLOGICAL SURVEY (November 12, 1957)

Reappraisal of networks

In 1955 the Geological Survey began a comprehensive study of the network of river measurement stations, chemical quality sampling stations, and sediment measurement stations in the continental United States, Alaska and Hawaii. These networks of stations together with the ground-water observations wells constitute the Survey's continuing program of systematic measurements and observations of the occurrence, movement and quality of the water resources of the nation. The study was designed to determine the effectiveness of the network in defining the areal variations in the factors being measured and to serve as a guide in making adjustments and improvements to help meet current and anticipated needs for data.

The network analyses now being carried on are merely one step in the continuing appraisal of the adequacy of the programs of collection of basic hydrologic data. The collection of basic data is intimately associated with, and in many respects an integral and inseparable part of, the total investigational

program which includes the processing, analysis and interpretation of the basic measurements and observations. This is particularly applicable to investigations of the character and availability of ground water. The determination of the geologic, hydrologic, and hydraulic characteristics of aquifers is a complex problem requiring close control of investigations and on-the-job analyses and interpretations of data. Networks of measurement stations therefore are only a part of the total objective of providing adequate knowledge of the water resources. The results of these network appraisals will be published in appropriate forms as promptly as practicable. The great amount of detailed work involved indicates a summary form of publication.

The stream gaging network is composed of three principal components. A primary network of long-term gaging stations, well distributed geographically and operated indefinitely, provides an index of the long-range fluctuations in streamflow for any region. A secondary network of relatively short-term stations defines the areal variations and characteristics of runoff; data from stations in this secondary network and other data collected on a short-term basis may be correlated with data from the permanent type primary network, thereby achieving perspective in relation to long-term climatic and hydrologic fluctuations. Stream measurement stations that provide data specifically needed for project operation or water management purposes comprise the third category. The primary and secondary networks are supplemented by measurements of flood and drought flows and other investigations as required. Some of the technical aspects of this network analysis have already been published.

The study of the stream gaging network is essentially complete, and reports for various states and regions are under preparation. While the study cannot be expected to provide a complete measure of the deficiencies in data and network requirements, it does contribute substantially to the appraisal of current needs for data and provides a guide for future network expansion. It indicates, for instance, that the existing primary network of 2,600 stations should be selectively expanded to about 3,300 stations, and that nearly 1,300 existing stations can be relocated after a period of operation of 5 to 10 years duration. The study also indicates that about 1,150 previously discontinued stations had accumulated sufficient records to permit adequate correlation with base stations. Also, 1,250 existing stations operated for water management purposes also qualify for the secondary hydrologic network. It was concluded that about 2,500 additional short-term secondary stations should be established promptly at other selected locations for a well-balanced and currently adequate stream gaging program. A total of about 10,000 active gaging stations is indicated as the desirable network size to meet present requirements. At the present time 6,640 stream flow measurement stations are being operated.

Daily records of sediment movement in streams are obtained at approximately 150 locations. Because the coverage is relatively sparse and large areas are inadequately sampled, the analysis of the sediment station network is less comprehensive than that carried out for the stream-gaging network. The study is not yet complete, but activity to date indicates that it will result in improved criteria for selecting (a) the optimum composition of a sediment program with respect to measurements, analyses, interpretation, and research at a selected level of activity; (b) the distribution of observations with respect to time and place; and (c) the characteristics and properties of sediment and its watershed environment that should be observed to maximize

he utility of the results. The design of the network is underway and other complementary studies will follow. At the present time the broadly stated goals of the President's Advisory Committee on Water Resources Policy with respect to sediment investigations appear applicable.

As of June 30, 1957, there were 300 daily chemical quality stations in operation. The appraisal of this network is being carried out on approximately the same basis as that for the sediment stations, being limited in scope because of the relative sparseness of the coverage. The analysis is expected to provide an objective, systematic approach to chemical quality investigations, and improved criteria for the establishment of stations comprising the network. The concept of diversity of chemical quality from one stream basin to another and from one geologic province to another is one of the major factors in this approach.

General and specific policies

The broad objective of the Geological Survey's program with respect to water resources is the determination and description of the quantity and quality of the surface and underground waters of the Nation. This broad objective encompasses the appraisal of the total water resources; the variations in the availability of water with time and place; the effects of long-term trends and climatic fluctuations; the effects of the works of man on water availability, including consumptive and non-consumptive uses; the movement of water through the permeable portion of the earth's surface; and the magnitude and frequency of floods and low stream flow.

For many years the water resources problems of the country were such that primary emphasis in the Geological Survey's program was placed upon the collection of basic water resources data. The increasing demand for solutions of increasingly complex water problems now necessitates greater attention to analytical, interpretative, and research phases of water resources investigations. The Nation is at a stage of development wherein a concerted attack is urgently needed upon all phases of water resources problems. The Survey recently began to increase the amount of work being done on those phases. Increased and intensified effort is also being directed toward research in the basic physical processes involved in the occurrence and movement of water in order to understand more fully the various facets of the hydrologic phenomena which we are observing and interpreting. Experimentation and additional research directed toward the development of improved techniques for investigations and analyses have been carried on and are now in progress by the Geological Survey. The investigations include study of the movement of water through unsaturated media, evaporation from free water surfaces, evapotranspiration, the effects of land use, sediment transport, channel geometry and river morphology, erosion, infiltration, and recharge. A better understanding of the physical processes in the interrelation of the various factors influencing the occurrence and movement of water facilitates more effective collection and interpretation of basic data. Consequently improved knowledge is available for application to the planning and design of water projects.

Adequacy of basic enabling legislation

The Act of March 3, 1879, creating the Geological Survey, did not define specifically its responsibilities in the water resources field. Annual

appropriation acts, beginning with 1888, have authorized the use of appropri- ated funds for water resources investigations. In the 1894 statute, Congres made funds available to the Geological Survey for "gauging the streams and determining the water supply of the United States, including the investigatio of under-ground currents and artesian wells in arid and semi-arid sections Successive appropriation acts have continued or further define the authoriza- tion for the water resource functions of the Survey. The current programs and any presently contemplated future recommendations are fully authorize in the language of the present appropriation act.

Federal cooperation with state and local agencies

For more than 50 years the mutual interests of the State and Federal governments in water resource problems have been implemented in the Geological Survey by cooperative investigations. These cooperative activiti have been a major part of the work of the Survey, and through them far mor basic data have been obtained than would have been acquired otherwise. Th current trend is for increased participation by states in cooperative water resources investigations. The Survey will continue to acknowledge this par- ticipation to the greatest extent possible within its available financial and manpower resources and in accord with the desirable intention of maintain- ing a balanced over-all program of mapping, investigations and research. Because the cooperative programs are geared substantially to water prob- lems of intra-state nature, the Federal interest requires additional investi- gations related to problems of a regional and national nature. With the in- crease in water development and use, water problems are becoming less localized and more regional in nature, requiring collective action on the pa of the States and increasing the need for studies of the water resources on broad geographic basis. This condition has emphasized the greater need fo funds not restricted to the cooperative program.

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GROUND WATER UTILIZATION, SUFFOLK COUNTY, L.I., NEW YORK[a]

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ABSTRACT

Ground water pumped from the aquifers underlying Suffolk County, Long sland, N. Y. for all uses amounted to more than 24 billion gallons in 1955. 'ater problems that may arise through constantly increasing withdrawals in-.ude: extensive sea water encroachment, marked depletion of storage, in-ıstrial contamination, and rises in water temperatures.

INTRODUCTION

Suffolk County, N. Y., one of the fastest growing counties in the United :ates, has an area of 920 square miles — about three-quarters as large as hode Island. As can be seen in figure 1, it occupies the eastern two-thirds Long Island, with Nassau County, another rapidly growing county, on the est. Sea water bounds the county on three sides, dividing it geographically ito three areas: the main body and two peninsulas, termed the North and outh Forks. These forks originate at Riverhead and extend eastward for bout 27 and 40 miles, respectively. Although creating a favorable environ-ent for agriculture and the fishing and vacationing industries, the adjacent ılt water bodies constitute a source of potential contamination of Suffolk ounty's ground-water supply. Suffolk County's rapid industrial development ıs been, in large measure, due to the abundance and quality of this supply ıd as continued industrial growth at an increasing rate is a strong possibili-, investigations of the hydrology and geology of Suffolk County in relation to ea-water encroachment are of utmost importance.

The purpose of this paper is to outline briefly the ground-water hydrology ıd geology of Suffolk County, to describe the factors involved in ground-water

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CONNECTICUT
LONG ISLAND SOUND
ATLANTIC OCEAN
SUFFOLK
NASSAU
QUEENS
KINGS
NEW YORK
NEW JERSEY
HUDSON RIVER
PECONIC BAY
NORTH FORK
SOUTH FORK
Scale
0 5 10 15 20 Miles

ilization, and to discuss the present and future consequences of ground-ater development.

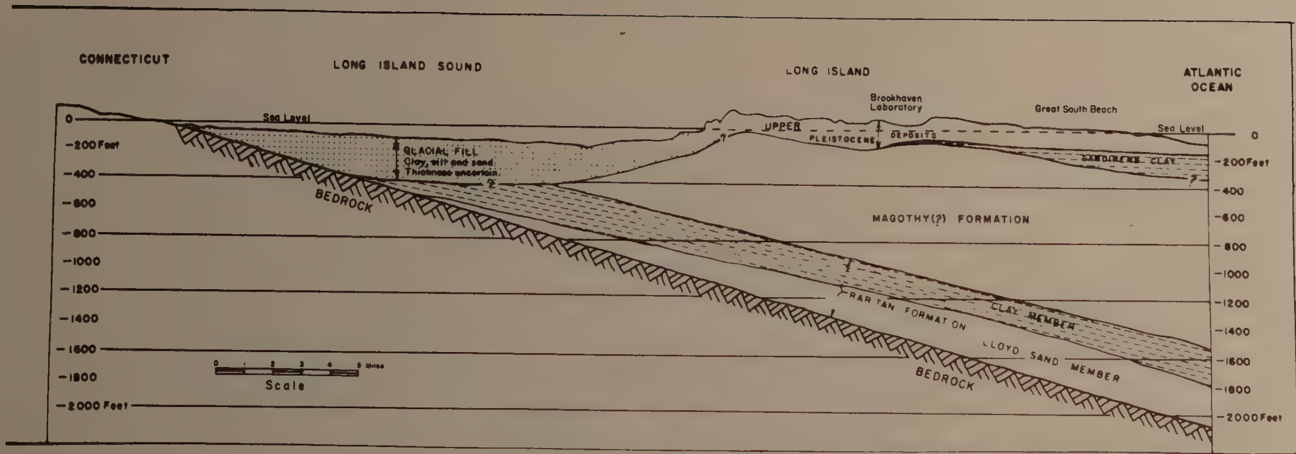
Ground-Water Occurrence and Movement

Ground water fills the pore spaces of the unconsolidated clays, sands, and ravels that underlie Suffolk County. A generalized geologic cross section is hown in figure 2. Three aquifers have been recognized. From bedrock up lese aquifers are the Lloyd sand member of the Raritan formation, the lagothy (?) formation, and the upper Pleistocene deposits.

The lithology of these aquifers is briefly described below. More detailed overage is made by de Laguna and Perlmutter (1949).

The Raritan formation, deepest of the unconsolidated deposits on Long sland, rests on a basement of crystalline bedrock. The Raritan is composed f two members - an upper clay member and lower Lloyd sand member. ach is conformable with the bedrock basement which generally slopes at bout 80 feet per mile in a southeasterly direction. The clay member beneath ost of Long Island usually consists of beds of silty and solid clay with oc-asional sandy layers. The Lloyd sand member, on the other hand, is a pre-ominantly coarse sand and gravel intercalated at intervals with thin layers f silt and clay. As the overlying clay member has a very low permeability, e water in the Lloyd sand member is for the most part confined under rtesian pressure. The Magothy (?) formation is considerably thicker than e underlying Raritan formation in most places on Long Island. It consists f layers of fine sand, silt, and clay interbedded with several zones of coarse and and gravel. Pre-glacial erosion of the Magothy (?) formation has caused considerable relief to be developed in the upper surface. Consequently, the epth at which this surface is first penetrated by wells is quite variable. The ater in the Magothy (?) formation is generally under artesian pressure, but cally it is unconfined. Glacial deposits of late Pleistocene age mantle the lder formations in practically all places on Long Island. They form the resent land surface and surficial deposits of Suffolk County. Two different pes of material are recognized from the cuttings of wells drilled into the eposits - till and glacial outwash. Till is a heterogeneous mixture of sand, lay and boulders deposited from glacial ice. It is generally poorly sorted nd the presence of clay, in some places, causes this material to have rather w permeability. In other places, however, the till is essentially free of clay nd quite sandy and locally may have high permeability. Outwash, on the her hand, was deposited from glacial melt water and is moderately to well orted. It is largely composed of sand with some gravel and has a relatively gh permeability. Stratification, however, and the presence of lenses of clay nd silt, interbedded with the sand and gravel usually causes the horizontal ermeability to exceed the vertical permeability.

The water table of Suffolk County, the upper limit of the entire thickness of aturated material, ranged in altitude from about 70 feet above sea level in e inland, central portion of Suffolk County to sea level near the shores. The tal amount of water stored below the water table amounts to many billions f gallons and depends chiefly on the volume of saturated material and the orosity. The aquifers are constantly receiving replenishment and are con-antly discharging. Variation in either replenishment, discharge, or both esults in changes in the amount of ground water in storage at any one time.



GENERALIZED GEOLOGIC CROSS SECTION SHOWING THE WATER-BEARING SANDS UNDERLYING SUFFOLK COUNTY, NEW YORK.

Fig. 2. Generalized geologic cross-section showing the aquifers underlying Suffolk County, N. Y.

Natural replenishment to Suffolk County's ground-water reservoir is de-
ved solely from precipitation, which averages about 43 inches per year. nly part of the precipitation reaches the water-bearing sands, for sizable nounts are lost by evaporation. Losses by overland runoff to streams in ndeveloped areas are small, because of the rapid infiltration of precipitation to the soil. It is estimated that, as a result of these factors, about 50 per-ent of the precipitation reaches the water table or an average of about 350 llion gallons of water per year.

Natural discharge from Suffolk County's ground-water reservoir takes ace mainly through the seaward movement of streamflow and ground-water nderflow. Streamflow is largely ground-water outflow through channels that tersect the water table. According to information furnished by R. M. awyer of the U. S. Geological Survey the average discharge by streamflow about one-quarter of the average ground-water recharge or more than),000 million gallons annually. This is about 3-1/2 times the quantity of round water pumped for all purposes in 1955. Likewise, an unknown but robably greater, quantity of ground water moving through the subsurface ands, gravels, and clays is discharged each year directly into the ocean and to streams below the gaging stations. A third water loss, also unknown, but ossibly sizable in amount, occurs through evaporation from surface-water odies, such as streams and ponds, and evaporation and transpiration from e marshy and low-lying areas that fringe the shoreline of Suffolk County.

These are the natural elements of ground-water conditions in Suffolk ounty. Ground-water development can markedly alter the natural pattern. ome of the changes have a beneficial result, others create problems.

Quality of Water

Water utilization, in general, requires water adequate in quality as well as quantity. Substantial amounts of dissolved solids in the water can cause a umber of problems. Scale in boiler tubes and piping, resulting from the de-osition of the carbonates of iron, calcium, and magnesium, impairs heat ansfer and increases head losses. Encrustation of well screens by these or her compounds reduce both the yield of producing wells and the ability of charge wells to take water. Sizable concentrations of iron and manganese lts stain laundry and water fixtures. Excessive concentrations of nitrate, lfates, and chlorides can produce organic disorders. Water with a pH less an about 6.0, especially in the absence of carbonates, has a marked corro-ve effect on metals, causing equipment failure and deterioration. Accepta-e standards for water quality differ depending on the particular use desired. general, a water supply that conforms to the chemical and bacterial stand-rds for drinking water is suitable for most purposes.

Figure 3 shows the locations of 15 water-supply plants in Suffolk County. 1955, these plants, including 25 separate well installations, supplied more an 6,000 million gallons of ground water for public supply, laboratory, and stitutional usage (see footnote - table 1). According to estimates of the ew York Water Power and Control Commission this pumpage is about one-arter of the total ground-water pumped in Suffolk County in 1955.

Table 1 shows the type of water treatment employed at these wells to sup-y water of satisfactory quality to the consumer. The data were obtained om the Chief Engineers or the Superintendents of the respective plants. No

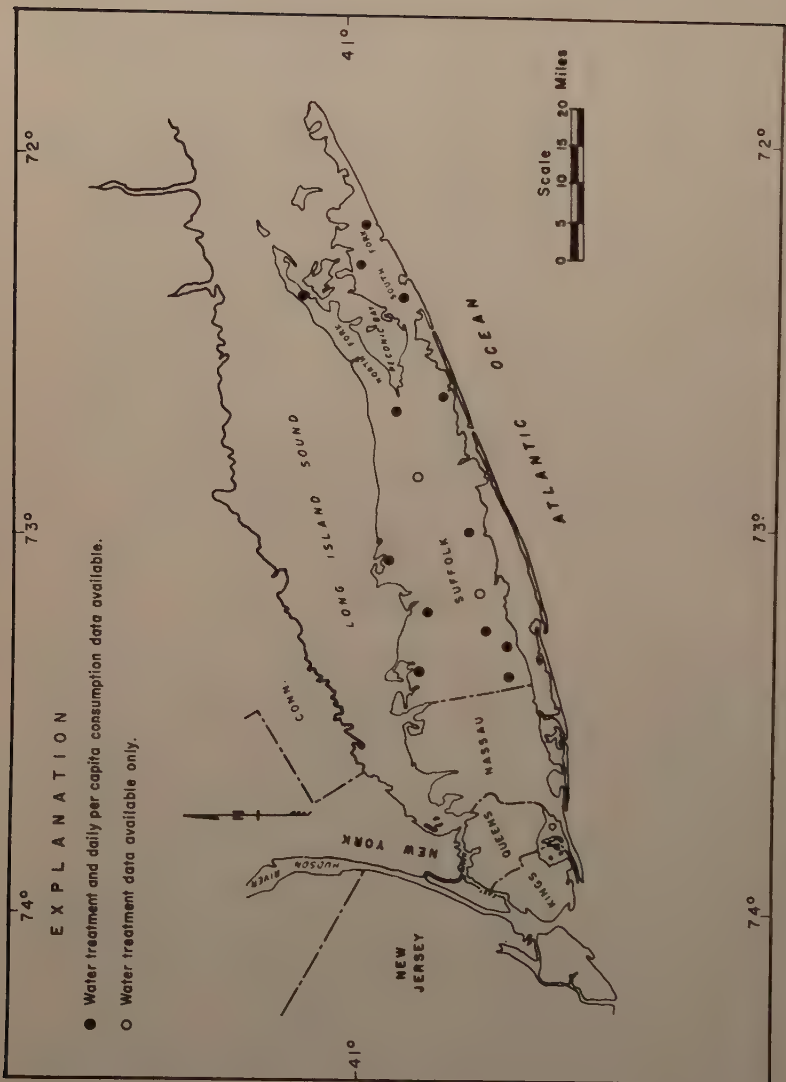


Fig. 3. Map of Suffolk County, N. Y., showing the location of water-supply plants for which data on water-treatment and daily per capita use

Table 1 - Treatment required for utilization of ground water in Suffolk County, N.Y.*

Well site location in Suffolk County	Owner	Water treatment required		
		None	Corrosion control	Chlorination
	Wells tapping the upper Pleistocene deposits			
Riverhead**	Town of Riverhead			
Greenport	Village of Greenport		x	x
Easthampton	Home Water Co.	x		
Upton	Brookhaven National Laboratory		x	x
Amityville (Greene Ave.)	Suffolk County Water Authority		x	x
North Amityville	do.		x	
West Babylon	do.		x	
Babylon (Smith St.)	do.		x	
North Babylon	do.		x	
Bayshore	do.		x	x
North Bayshore	do.		x	

See footnotes at end of table.

Table 1 - (Cont'd)

Oakdale	Suffolk County Water Authority	x		
Bayport	do.		x	
Central Islip	do.		x	
Bellport	do.	x		
Westhampton	do.		x	
Southampton	do.		x	
Sag Harbor ***	do.		x	x
Smithtown **	do.			x
Centerport	do.	x		
	Wells tapping the Magothy(?) formation			
Central Islip	Central Islip State Hospital		x	x
Cold Spring Harbor	Suffolk County Water Authority	x		
Port Jefferson	do.	x		

See footnotes at end of table.

Table 1 - (Cont'd)

	Wells tapping both the upper Pleistocene deposits and Magothy(?) formation		
Kings Park	Suffolk County Water Authority	x	
	Wells tapping both the upper Pleistocene deposits and the Lloyd sand member of the Raritan formation		
Huntington (Mill St.)	Suffolk County Water Authority		x

* Usage:

Brookhaven National Laboratory - for general use and public supply.
Central Islip Hospital - for institutional use.
All others listed - for public supply

** Odor control also included in treatment.

*** Iron and turbidity control also included in treatment.

treatment whatsoever is necessary at six installations. Corrosion control is necessary at sixteen well installations and is the only treatment widely used. Chlorination at the present time is largely a voluntary precautionary measure rather than a remedial measure required by the State Health Department. Treatment for iron, turbidity, and odor is needed only in isolated cases. Because of the low concentration of carbonates and bicarbonates and of magnesium and calcium ions water softening is unnecessary.

Besides chemical purity, water used for drinking, cooling, and air-conditioning must have a relatively low temperature. Here again, Suffolk County is fortunate, for in general, ground-water temperatures range from 50°F to slightly more than 64°F.

Ground-Water Utilization

In 1955, according to estimates of the New York State Water Power and Control Commission, more than 24,000 million gallons of ground water were utilized in Suffolk County (Anon., 1956). This represents about 7 percent of the average ground-water recharge in the area. Most of the withdrawals in Suffolk County are from the upper Pleistocene deposits and the large volume of water in storage in the deeper Magothy (?) formation and the Lloyd sand member of the Raritan formation is relatively untapped.

Methods of withdrawal

More than 15,000 wells have been drilled, driven, or jetted into the water-bearing sands underlying Suffolk County. For the most part, these range in depth from a few feet to about 800 feet and have steel casings ranging in diameter from 1-1/4 inches to about 12 inches. Well screens which are a necessary part of the well construction range in length from 2-foot well-poi[nt] screens on domestic wells to 75-foot (or more) screens on major public-supply wells. An important part of the well drilling operation is well develo[p]ment, which involves the removal of the finer-grained particles from aroun[d] the screen. Were it not for well development, many wells screened in unco[n]solidated material could be pumped only at very low rates.

Where the depth to water is less than about 8 feet, artificial ponds are oc[-] casionally used as a source of irrigational water. The dimensions of the ponds average about 40 feet square and 6 feet deep but differ considerably f[or] individual ponds.

Deep-well turbine pumps are the most commonly used type in Suffolk County. Centrifugal pumps, although used to some extent, are limited by their inability to raise water continuously from depths greater than about 1[5] feet. A modification of the deep-well turbine pump that has recently come [into] popular use is the submersible pump. All the working parts of this type pu[mp], including the electric motor, are contained inside the well, being submerge[d] below the water level. Where small-capacity pumps are required, such as for household use, and where the depth to water is beyond suction lift, jet pumps are frequently used.

Pumps for irrigation wells are usually driven by gasoline or diesel engines. Pumps for public-supply, industrial, and domestic wells are usually driven by electric motors for smoothness and efficiency of operation. Most public-supply and large-capacity industrial wells are equipped with a gasol[ine] or a diesel engine for stand-by use in case of power failure.

ublic-Supply Usage

Withdrawals for public supplies, which accounted for about three-tenths of ie total pumpage in 1955, are made at 27 municipally-owned and about 71 rivately-owned water-supply plants. The largest system is that of the iffolk County Water Authority which supplied about three-fifths of the ground ater used for public supply. Water pumped for public supply is used for a ariety of domestic, industrial, and agricultural uses. In eastern Suffolk ounty the uses are predominantly domestic and irrigation; in western Suffolk ounty the uses are domestic and industrial.

Daily water use per capita is an approximate measure of the degree of de- elopment of a community. In some areas development may be due to indus- y, in others, to agriculture. Community growth, both in size and wealth, esults in increased water use owing to the large number of water-using de- ces. For purposes of this paper, figures for daily use per capita in Suffolk ounty were obtained by dividing the total plant pumpage in an area by the imber of users served by that plant. New York City, a highly commercial- ed and industrial community, has a daily use of about 130 gallons per capita. rural community with few water-using devices may have a daily use as low s 25 to 30 gallons per capita.

Listed in table 2 are the 1955 daily use per capita for some water-supply ants in Suffolk County. The locations of these plants are given in figure 3. igures shown in table 2 are based on data obtained from the Chief Engineers the Superintendents of the plants involved. The highest daily use of 126 illons per capita was in the Incorporated Village of Greenport, a developed, ewered community. Here, more than one-tenth of the pumpage is used for rgation of crops. The lowest, 42 gallons, was in the evirons of the village Patchogue, a relatively undeveloped community. The average use per ipita for all the water-supply plants tabulated was 76 gallons.

More than three-eights of the people of Suffolk County depend entirely upon rivate domestic wells for their supply. As Suffolk County is developed do- estic wells probably will be replaced gradually.

Both the installation of sewers and growth of population in an area tend to crease the water use per capita. Consequently, estimates of the future ater-supply needs of Suffolk County based on present-day use per capita may seriously in error.

dustrial Usage

Withdrawals from private industrial wells supplied about one-third of the tal ground water pumped in Suffolk County in 1955. These withdrawals rgely took place in the western half of Suffolk County, being used by hos- tals, aircraft factories, food-processing plants, dairies, laundries, light anufacturing plants, storés, offices, and public utilities other than water pplies.

About two-thirds of the water pumped for industrial purposes is used for oling including air conditioning. As cooling does not alter the chemical ntent of the water, the State conservation law requires that the spent water recharged to the source formation. This is commonly done through wells, lled diffusion wells, and through excavated pits, called recharge basins.

Table 2.- Daily use per capita for selected water-supply plants in Suffolk County, N. Y., in 1955.

Plant location	Owner	1955 Pumpage (millions of gallons)	Daily use per capita (gallons)
Greenport	Village of Greenport	166.1	126
Easthampton	Home Water Co.	213.4	104
Huntington	Suffolk County Water Authority	1,014.6	95
Westhampton	do.	155.3	87
Port Jefferson	do.	243.4	81
Southampton	do.	142.0	73
Bayshore	do.	1,155.9	72
Riverhead	Town of Riverhead	230.1	70
Babylon	Suffolk County Water Authority	598.2	68
Amityville	do.	366.9	67
Sag Harbor	do.	62.0	66
Smithtown	do.	106.2	60
Kings Park	do.	70.2	53
Patchogue	do.	376.4	42
Average			76

Agricultural Usage

The third important use of ground water in Suffolk County is for irrigati In 1955, one-quarter of the total ground-water withdrawals was used to irri gate about 56 square miles of intensively farmed land, about one-third of al the farmland in Suffolk County. These withdrawals were made during a 107 day period starting in the middle of June.

Since the end of World War II, acreage irrigated by ground water in Suff County has more than quadrupled. The most intensively irrigated area of Suffolk County is the northeastern corner, north of the town of Riverhead ar extending along the entire North Fork.

Figure 4 shows a typical irrigation-well installation. The pump which is mounted over the well on a concrete base, is driven by direct drive. Figure

Fig. 4. Deep-well turbine pump used to supply irrigation water.

hows the method by which well water is applied. Water is pumped through ortable aluminum pipe to oscillating sprinklers which spray it in a circular attern. Sprinklers differ widely in capacity and design. The type shown in ie figure distributes about 15 gallons per minute. Another type can dis-ribute 400 gallons per minute through one sprinkler head.

The amount of water used and the frequency of application vary widely. ome farmers irrigate when the soil no longer cakes upon being squeezed. thers follow the recommendations of the Long Island Research Farm at aiting Hollow and supplement the rainfall with enough water to insure the pplication of 1 inch of water per week to the land. Still others, who have few ieces of equipment and are short of manpower, adjust their irrigation pro-edure accordingly.

No data are available but it is thought that a large percentage of the irriga-on water applied to crops in Suffolk County is evaporated from soil and plant urfaces. At times excessive amounts of water may be applied to the soil esulting in the return of part of the water to the water table. In general, owever, probably only a small part of the water pumped for irrigation is eturned to the water-bearing deposits.

The rapid increase in irrigation in the last 5 years is evidence of the re-ılts obtained. The farmer not only protects his crop investment during dry ears, but also increases the yield per acre in years of normal rainfall.

Fig. 5. Oscillating type of sprinkler used for crop irrigation.

Consequences of Utilization

Present Problems

At present, the chief problems caused by ground-water utilization in Suffolk County are sea-water contamination, and corrosion due to the use of untreated low pH water. Once full-scale development of the ground-water reservoir begins, additional problems may become important.

Sea-water contamination has been observed only in isolated wells in near shore areas or in other areas underlain by salty water at shallow depth (Hoffman and Spiegel, 1958). These areas include parts of the North and South Forks, off-shore barrier beaches such as Westhampton Beach and Fir Island, and a narrow zone, of undetermined width, adjacent to and parallelin the shoreline of Suffolk County, where the water table is only a few feet abov sea level. At some places wells had to be abandoned. Perhaps, the most critical contamination in Suffolk County is that of the well-water supply of th village of Greenport. This village, population 3,000, near the eastern tip of

he North Fork, utilizes about 130 to 160 million gallons of water annually. At 3 of the 4 well installations, the water pumped has chloride concentrations narkedly above normal. During times of peak demand the water pumped at 2 f these stations has chloride concentrations of more than 400 parts per milion, considerably above the maximum of 250 parts per million recommended y the U. S. Public Health Service. Only be adept combination of this water vith water of low chloride concentration from the fourth pumping station can potable water supply be delivered to the user.

As previously shown, much of the ground water of Suffolk County is corosive. Large water-supply installations can cope with this problem eco-omically; however, corrosion treatment for individual well systems is not lways economically feasible. As a result, the equipment and piping of many f these systems must be replaced ultimately. Although no specific dollar alue can be assigned, the resultant financial loss can be considerable.

'uture Problems

Future problems arising from increased ground-water utilization in Sufolk County can only be conjectured. Some of the more serious of these are: 1) more extensive sea-water contamination of the ground-water reservoir; 2) contamination of the ground water by industrial wastes, cesspool effluent, nd fertilizers; (3) increases in ground-water temperature due to artificial echarge; and (4) permanent depletion of ground-water storage, especially if uffolk County should be extensively sewered.

Heavy ground-water withdrawals, particularly in shoreline areas, can ause extensive sea-water contamination of the ground-water reservoir. The nagnitude of the allowable withdrawals is determined by many factors, and a etailed appraisal of ground-water conditions is required before any conlusions can be reached. In near-shore areas or in areas where the fresh vater is underlain by salt water even moderately heavy development may ause encroachment. On the other hand, extremely heavy withdrawals in the entral inland portions of Suffolk County may produce no measurable enroachment whatsoever.

Ground-water contamination can also result from the recharge of wastes. n future years, without remedial measures, wastes of all kinds may be reharged to the water-bearing sands faster than they can be discharged. As a esult the concentrations of certain compounds will increase. Persistent ontamination from this source ultimately might create local, if not countyvide, health problems.

At the present time, contamination by high-chloride fertilizers is indicated n the intensely cultivated areas of Suffolk County (Hoffman and Spiegel, 1958). s the history of this contamination is largely unknown, very little can be redicted concerning its progress in future years. However, the concentraon of nitrates and chlorides in the ground water possibly may increase sufciently to render the water unpalatable or to cause organic disorder.

Changes in the temperature of ground water can be as important as chemial contamination. The recharge of spent cooling and air-conditioning water varms up the ground water in the vicinity. The recharge of small volumes of lightly warmed water creates no problem, whereas the recharge of large olumes of high-temperature water does, and in intensively developed indusrial areas this condition could result in costly cooling problems as happened t one locality in western Long Island, N. Y., where well-water temperatures s high as 90°F have been measured (Brashears, 1946).

Permanent and sizable depletion of ground-water storage in Suffolk Count may result from extensive sewering. The only sewers in Suffolk County at the present time are the small systems of the near-shore villages of Greenport, Huntington, Northport, Ocean Beach, Patchogue, Port Jefferson, and Riverhead. So, probably 80 percent of the water pumped from public, privat and industrial wells is returned to the ground. If the area served by sewers expands, and if storm water is discharged directly into the ocean, replenishment previously obtained from sanitary wastes and storm water will be considerably reduced. On the other hand, withdrawals from the ground-water reservoir will not lessen and the increased net draft on the ground-water wi cause a decrease in ground-water storage and a decline of the water table.

One consequence of appreciable water-level decline would be the need to deepen some of the existing shallow wells for further use. Another would be an increase in pumping costs incurred by the additional lift required to raise the water to land surface. Perhaps of greatest consequence is the possibilit of a landward movement of sea water that would cause the complete abandonment of pumping installations.

An example of the problems that can arise from intensive development is in Kings County, the westernmost county of Long Island, New York. Here, the population density is more than 36,000 people per square mile. Ground water recharge is greatly reduced by paved surfaces and the sanitary and storm sewers that drain the area. Heavy pumping by the New York Water Service Corporation, near the geographical center of the county, lowered water levels at some places to more than 20 feet below sea level. The resul ing sea-water encroachment raised chloride concentrations in the adjacent ground water to well above 1,000 parts per million, and the system was shut down in June 1947 (Lusczynski, 1952). At the present time, the water supply for this area is imported from upstate sources.

Remedial Measures

The problems described above can be costly. Possible remedial measures might include: (1) good water budgeting; (2) the proper location of ground-water withdrawals; (3) the removal of wastes by sewers; (4) constan supervision and evaluation of ground-water conditions; and (5) foresight in the planning of ground-water development.

Good water budgeting serves to increase ground-water replenishment while decreasing ground-water losses. If present growth trends persist, at some time in Suffolk County's future, seaward disposal of sewered wastes will probably be required. Consequently, concurrent with sewer installation measures to offset ground-water depletion should be considered. These might include properly located storm-water recharge basins and, if acceptable, sanitary-waste leaching basins. Recharge near the shore of a selected part of the sewage would help to buffer sea-water encroachment. Another budgeting measure to offset ground-water depletion would be to reduce the natural ground-water discharge by streamflow, underflow, and evaporation. Wells near the shore or next to streams, pumped at appropriate rates, would capture flow now being lost seaward. Backfilling marshy and lowlying areas would reduce evaporation and transpiration losses and at the same time make the land available for development.

Careful selection of well-sites and regulation of ground-water withdrawals would help to minimize sea-water encroachment. On one hand, wel

sites, should be located as far from the shoreline as possible. On the other hand, to intercept natural discharge, they should be placed near the shore. Some adjustment is possible in the type of installation constructed.

The complex interrelationships of the geologic, hydrologic, hydraulic, and socio-economic factors involved in the development of Suffolk County's ground-water reservoir requires careful evaluation of the problem. At the present time part of this evaluation is being undertaken by the U. S. Geological Survey. In cooperation with local agencies, the Geological Survey is appraising the occurrence, movement, quality, and quantity of the ground water underlying Suffolk County, in addition to investigating conditions which might ultimately threaten its quantity or quality.

The New York State Water Power and Control Commission, which has authority to regulate large-scale uses, except agricultural, supervises reservoir development. This regulatory body licenses well drillers, and determines policies concerning ground-water withdrawals and artificial recharge.

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DESIGN METHODS FOR FLOW IN ROUGH CONDUITS[a]

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ABSTRACT

Design curves and methods are presented for determining friction factors for turbulent flow in closed conduits and tranquil open channel flow. The methods are physically realistic, being based on five distinct regimes of turbulence and related to the actual physical dimensions of the boundary roughness elements.

One of the most basic and frequently occurring engineering problems is that of the conveyance of water or other fluid by means of some sort of flow channel. The channel may be either closed or open, the basic mechanics being essentially the same. Usually the conduit lining is hydraulically rough and the flow is turbulent.

Despite the commonplace nature of this problem, design methods for dealing with it are still largely empirical. Formulas such as those of Manning, Kutter, Scobey, Hazen-Williams and many others are illustrations of this fact. Each of these formulas is dimensionally incorrect and has a relatively limited range of physical application. Each applies only to the turbulent flow of water, at relatively high Reynolds Numbers, and assumes uniform flow conditions. But their most serious defect is the wholly empirical nature of the provision made for the effect of boundary roughness. It has often been shown that these so-called "roughness coefficients" not only vary tremendously with the specific nature of the conduit surface, but frequently vary also with the Reynolds Number, hydraulic radius, and other factors.

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a. Paper presented at 7th National Conference, ASCE Hydraulics Division, at Atlanta, Georgia, Aug. 21, 1958.

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It is well-known that dimensional requirements establish the correct flow function to be of the form known as the Darcy equation:

$$\frac{H_f}{L} = f \frac{V^2/2g}{D} \tag{1}$$

where H_f/L represents the unit expenditure of flow energy to overcome boundary friction, and $\frac{V^2/2g}{D}$ is the ratio of velocity head to equivalent diameter of conduit. The difficulty in the use of this simple and universal formula is of course in the evaluation of the dimensionless friction factor f, which can be shown to be, in general, (assuming uniform flow), given by the function:

$$f = F\left(N_R, N_F, N_W, N_M, \frac{h}{D}, \frac{\lambda}{D}, \frac{s}{D}, \frac{r}{D} - - - -\right) \tag{2}$$

The dimensionless force ratios, N_R, N_F, N_W, N_M represent respectively the influence on the flow structure of the forces of viscosity, gravity, surface tension, and fluid compressibility. For most problems in civil engineering hydraulics, the last two have negligible effect and can be eliminated from the function. Gravity has no direct effect in closed conduit flow and is often negligible in problems of tranquil open-channel flow. It will be assumed herein that this is the case, so that the following discussion will not apply quantitatively to rapid flow in open channels. The Reynolds Number, N_R, however, must nearly always be considered, since every flow phenomenon necessarily involves an expenditure of flow energy in friction. The other terms in the function reflect the influence of the wall roughness elements on the flow and turbulence patterns. Thus h, λ, s, r, represent the radial heigl longitudinal spacing, peripheral spacing and radius of curvature of the rough ness elements. As many similar additional terms should be included in the function as necessary to describe completely the geometry of the roughness elements.

Because of the complex nature of the friction factor function, various attempts have been made to group all the roughness dimensions together into a single representative dimension for simplicity. Since, however, each of the various dimensions may have a significant effect on flow geometry, this representative dimension usually cannot be any actual dimension, and must ther fore be determined empirically. It thus becomes little more than another empirical roughness coefficient. Furthermore there is no real assurance that such a representative dimension determined in this fashion would not al vary with Reynolds Number, hydraulic radius, etc.

The current widespread adoption of the so-called "equivalent sand-grain diameter" as the measure of roughness is a case in point. This concept, based on the well-known Nikuradse tests(1) on sand-coated pipes, has been highly popularized by the inclusion of the "Moody curves" in nearly all modern fluid mechanics textbooks.(2)

These curves, which give the friction factor as a function of the Reynolds Number and the ratio of equivalent sand roughness to pipe diameter, provide a compact design tool for use in flow problems. However, several serious logical fallacies are implicit in this procedure, as follows:

1) The Moody curves and the empirical values of equivalent sand roughne used with them are based on the Colebrook-White equation,(3) which in

turn is merely that of an artificial curve asymptotic to the Von Karman-Nikuradse so-called "smooth" and "rough" pipe curves. This is a transition function, showing the friction factor to be a decreasing function of the Reynolds Number. The artificiality of this device is evident, since the actual experimental transition function of Nikuradse, on whose results this procedure is based, is exactly opposite in character to the Colebrook function, showing the friction factor as primarily an increasing function of Reynolds Number. If there were really anything "equivalent" about Nikuradse's sand roughness, it should at least show the same form of functional relationship as obtained on the commercial pipes designed on the basis of it.

2) The Colebrook-Moody curves make no provision for conduit surfaces producing a rising friction factor-Reynolds Number relation (such as corrugated metal pipes[4] or flumes, sand-lined channels and conduits, etc.) or those with a fully horizontal characteristic[5] (such as conduit surfaces of so-called "random roughness").[6]

3) Even for surfaces which yield descending f-N_R curves, there are many data indicating that the Colebrook equation is not an adequate description of this relation. Some tests on large conduits show that friction factors tend to decrease up to much higher values of Reynolds Number than predicted by the Colebrook relation.[7] Certain tests have yielded systematic variations in empirical values of equivalent sand roughness for a given surface,[8] proving it to be unreliable as a means of predicting friction factor unless these variations are taken into account. Various data are available[9] refuting the Colebrook Equation implication that friction factors for a given surface type always decrease with increasing diameter.

4) Perhaps the most serious objection to the use of the "equivalent roughness" concept is that its superficial rationality tends to give the designer an unwarranted confidence in the accuracy of its predictions. The large number of roughness dimensions influencing the flow turbulence indicates the impossibility of representing the element by any single dimension, - especially some non-existent "equivalent" dimension, - and the several types of possible f-N_R relationships indicate the impossibility of using any single function to define this relationship.

Regimes of Turbulent Flow

A careful consideration of the mechanism of production and dissipation of turbulence will show that there are several distinct regimes of turbulent flow possible in conduits, each with its own peculiar characteristics. Turbulent flow is characterized by the continual generation and dissipation of vortices, with most of the energy loss occurring during the process of generation, the remainder in the process of viscous dissipation. In general, the generation of vorticity is a viscous phenomenon resulting from fluid moving at relatively high velocities in contact with fluid moving at relatively low velocities. The phenomenon typically occurs either at a laminar boundary layer or at a zone of separation behind a roughness element.

In either case, the vorticity so generated washes out into the main body of flow. The processes of mass and momentum transfer, as well as viscous

attrition, combine to reduce the turbulence structure to a certain typical pattern in the interior region of the conduit, regardless of the method by which the original vorticity was generated.

This "normal" turbulence can be treated statistically, yielding the well-known velocity distribution equation:

$$\frac{v_c - v}{v_s} = \frac{1}{k} \log_e \frac{r_o}{y} \tag{3}$$

In this equation, $\underline{v}$ is the velocity at distance y from the wall, $\underline{v_c}$ is the center velocity, at distance r_o from the wall and v_s is the "shear velocity," determined by wall shear stress. The constant "k" is the von Karman universal turbulence constant characteristic of this "normal turbulence." Equation (3), if plotted on semi-logarithmic paper, yields a straight line, of slope 1/k. It is assumed, of course, that the section under discussion is sufficiently removed from the inlet or other disturbance to permit full development of the turbulent boundary layer.

However, normal turbulence will usually prevail only in the central regions of the conduit. Near the wall, the velocity distribution will be materially affected by the nature of the vortex-generating mechanism. If the wall is smooth, with a laminar boundary layer, the very small-scale vorticity being generated, together with the transitional effects from laminar to turbulent flow, causes a steepening of the velocity gradient for some distance from the wall.

On the other hand, if the wall has roughness elements which pierce the laminar film, large-scale vortices are continually generated in the separation zones behind each element. This is manifested by a relative flattening of the velocity gradient for a region near the wall. These various velocity distribution phenomena are illustrated schematically in Figure 1.

On a conduit surface with only occasional roughness elements, both these effects would be combined. It becomes obvious, then, that the longitudinal frequency of vortex-generating roughness elements is of determinative significance with reference to the regime of turbulence that will prevail. With these preliminary considerations in mind, several regimes of turbulent flow in conduits may be considered.

Smooth Turbulent Flow

If the conduit wall is smooth, so that the turbulence-generating mechanism is solely one of minute vorticity shed from the laminar boundary layer, then the velocity distribution and friction factor are independent of all wall roughness dimensions. They depend rather on the thickness of the boundary layer, which is a decreasing function of Reynolds Number. The velocity distribution equation is:

$$\frac{v}{v_s} = \frac{1}{k} \log_e \frac{y v_s}{\gamma} + A \tag{4}$$

The constant A is empirically determined to be 5.5 if the turbulence constant $\underline{k}$ is taken at its usually accepted value of 0.4. The corresponding friction equation, showing the friction factor to be a decreasing function of Reynolds Number, is:

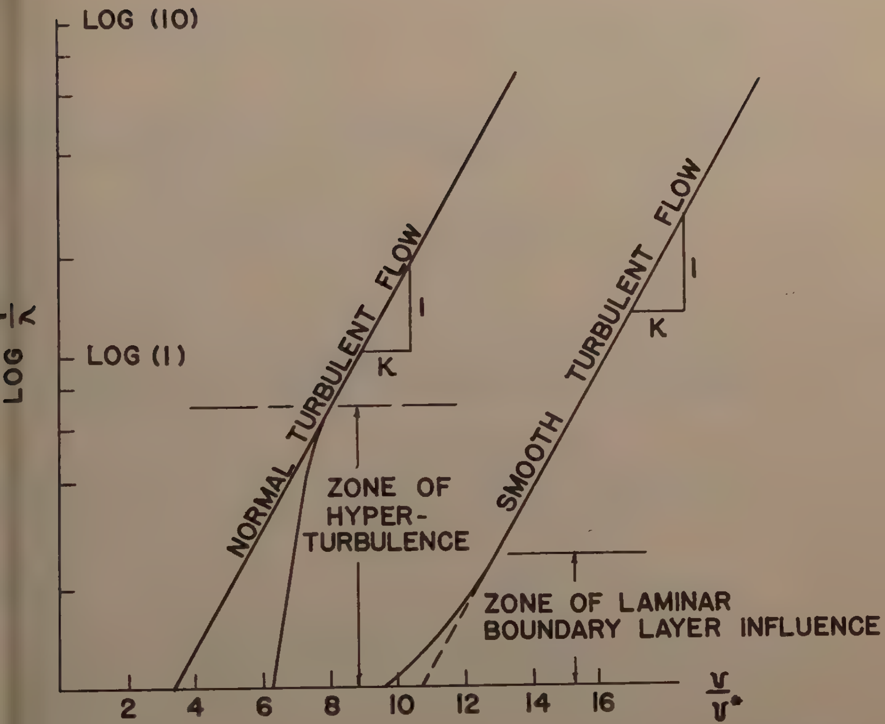


FIGURE 1

DIMENSIONLESS VELOCITY DISTRIBUTIONS FOR TURBULENT FLOW

$$\frac{1}{\sqrt{f}} = 2 \log_{10} N_R \sqrt{f} - 0.8 \tag{5}$$

This equation was derived for closed conduits. The corresponding smooth turbulence equation for open channels(10) is:

$$\frac{1}{\sqrt{f}} = 2.62 \log_{10} N_R \sqrt{f} - 3.16 \tag{6}$$

However, equation (5) may be used for open channels without serious error, at ordinarily high Reynolds Numbers.

Normal Turbulent Flow

Normal turbulence is defined as that that characterized by the universal turbulence constant $\underline{k}$. Normal turbulent flow would therefore involve a wall-to-wall homogeneity of the turbulence conforming to equation (3). There

could be no wall zone of either sub-normal or abnormal turbulence. Vortices generated at the wall are almost immediately broken up into patterns statistically equivalent to the normal turbulence of the central region.

The velocity distribution equation for normal turbulent flow may be written

$$\frac{v}{v_s} = \frac{1}{k} \log_e \frac{y}{\lambda} + B \tag{7}$$

Here λ is the longitudinal spacing of the roughness elements, which is the roughness dimension characterizing the frequency of wall vorticity sources (it may be noted in passing that λ is numerically equal to the sand-grain size in the special case of uniform sand roughness as used by Nikuradse). In normal turbulent flow the other roughness dimensions are irrelevant, except that the crests of the elements determine the physical limits within which the velocity distribution equation is applicable. Whatever shape the elements may have has no effect on the turbulence pattern, which is fixed in terms of the constant k.

Experimental work has yielded a value of about 8.5 for the constant B, for various types of roughness elements. The value of about 7.8 seems somewhat better for two-dimensional closed flow or rectangular open channel flow. In either case the corresponding friction factor equation(11) is:

$$\frac{1}{\sqrt{f}} = 2\log_{10} \frac{r_o}{\lambda} + 1.75 \tag{8}$$

Here r_o is the pipe radius, or equivalent radius in the case of a non-circular pipe or open channel. In this type of flow, the friction factor is independent of both the Reynolds Number and the type of roughness, depending solely on the relative roughness spacing. Note that the friction factor is defined in terms of the actual roughness dimension, not some "equivalent roughness."

Semi-smooth Turbulent Flow

If an ordinarily smooth conduit surface is interspersed with occasional isolated roughness elements, the over-all friction factor will be that due to the friction drag at the laminar boundary layer plus that due to the form drag forces on the roughness elements, a phenomenon illustrated in Fig. 2. The following equation(12) results:

$$f = f_s \left(1 + 67.2\, \Sigma E\right) \tag{9}$$

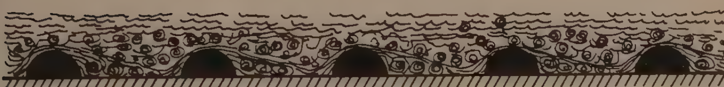
where f_s is the smooth turbulence friction factor at the given Reynolds Number, and E is an element characteristic defined as follows:

$$E = C_D \frac{p}{P} \frac{h}{r_o} \frac{r_o}{\lambda} = C_D \frac{p}{P} \frac{h}{\lambda} \tag{10}$$

p/P is the peripheral roughness ratio (the ratio of total peripheral length of roughness elements to total conduit wetted perimeter). h/r_o is the relative roughness height, r_o/λ has already been termed the relative roughness spacing, and h/λ is the roughness index. The coefficient C_D is the drag coefficient for the particular element shape, determined as for an airfoil

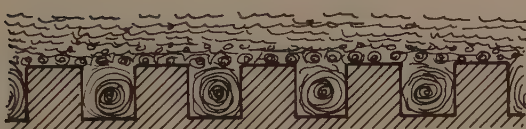
SEMI-SMOOTH TURBULENT
(ISOLATED-ROUGHNESS) FLOW

FIGURE 2



HYPER—TURBULENT
(WAKE-INTERFERENCE) FLOW

FIGURE 3



QUASI-SMOOTH (SKIMMING) FLOW

FIGURE 4

shape by the streamline configuration bounding the zone of separation.

A value of E can be computed for each repeating type of roughness element on the surface. The individual values are added to get the sum effect. Equation (9) indicates the friction factor relation for semi-smooth flow to be similar to that for smooth flow, with the friction factor computed as the smooth turbulence friction factor multiplied by a number greater than unity, the value of the number being $(1 + 67.2 \ \Sigma E)$. The friction factor for semi-smooth turbulent flow therefore normally decreases with increasing Reynolds Number and with decreasing values of ΣE.

Hyper-Turbulent Flow

If the wall roughness elements are sufficiently close together, the wake behind each may extend to or nearly to the next element. There is then essentially no part of the wall over which a laminar boundary layer exists. Furthermore the vortex generation and dissipation phenomena associated with each wake will interfere with those at the adjacent elements, so that the individual effects are not additive as in the case of semi-smooth flow.

The over-all phenomenon of wake interference results in a zone near the wall of abnormally intense turbulence and mixing. The velocity distribution

will be normal in the central regions, but the average slope near the wall will be somewhat flatter than normal, indicating a higher degree of turbulent mixing in this zone. This type of flow may be called hyper-turbulent flow, and is illustrated in Fig. 3.

The friction factor equation for this type of flow can be shown(13) to be the following:

$$\frac{1}{\sqrt{f}} = 2 \log_{10} \frac{r_o}{\lambda} + 1.75 + \phi \left(\frac{N_R \sqrt{f}}{r_o/\lambda} , \text{ element shape} \right) \qquad (11)$$

The function ϕ can be shown to decrease with increasing Reynolds Numbers, causing the equation (11) to approach that for normal turbulence, equation (8). The parameter $\frac{N_R \sqrt{f}}{r_o/\lambda}$ is a "wall Reynolds Number," equal to $\sqrt{32}\ \frac{v_s \lambda}{\gamma}$. The function ϕ may be regarded as a transition function whose magnitude decreases.

However, this function is not applicable for the entire transition from smooth to normal turbulent flow. Equation (11) indicates the friction factor f to increase the increasing Reynolds Number, whereas for both smooth and semi-smooth flow the friction factor decreases with Reynolds Number. It seems likely that for surfaces which will yield hyper-turbulent, and ultimately normal flow, there is a brief regime of smooth and then semi-smooth flow before hyper-turbulent flow begins. This sequence is evidenced by the familiar dip-and-rise of the Nikuradse rough pipe transition function.

Hyper-turbulent flow is produced over such surfaces as corrugated metal, sand-coatings and other surfaces of considerable roughness. The exact form of the transition function depends on the shape of the wake zones behind the elements and therefore on the form of roughness elements, and must thus far be determined experimentally. It is very significant, however, that for sufficiently high Reynolds Numbers, each transition function approaches the normal turbulent flow equation, for which the friction factor depends solely on the relative roughness spacing.

Quasi-Smooth Flow

Still another type of flow must occur over surfaces composed either of small depressions or of roughness elements spaced so closely as to form a more or less smooth pseudo-wall composed of the element crests and the enclosed pockets of dead fluid. Within these pockets or depressions will be stable vortices, unable to separate and commingle with the bulk flow because of the closeness of the downstream wall of the element. The depression vortices will be maintained through transmission of shear stress from the flowing fluid at their upper limbs. In addition, small-scale vorticity will be generated continuously along the pseudo-wall somewhat analagously to the process in smooth turbulent flow. Fig. 4 illustrates the phenomenon, which is herein called "quasi-smooth flow."

The energy expenditure is partially to generate the "quasi-smooth" vorticity at the pseudo-wall and partly to maintain the stable depression vortices. An approximate function(14) for the friction factor in this type of flow is as follows:

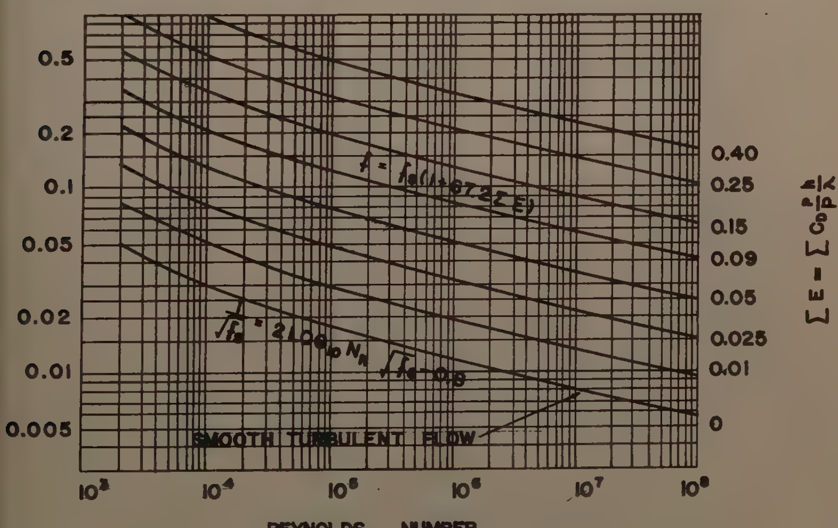
$$f = f_s + \Sigma \frac{p}{P} \frac{j}{\lambda} \left(\frac{c_w v_w}{V} \right)^3 = f_s + \Sigma X \qquad (12)$$

n equation (12), j is the longitudinal dimension of the depression, v_W is the elocity near the pseudo-wall, and c_W is the coefficient less than unity such hat $c_W v_W$ is the peripheral velocity of the depression vortex. Other terms ave been defined previously. Examination of the quasi-smooth flow equation eveals that the friction factor will normally decrease with increasing Reyolds Number.

Design Curves for Flow Regimes

In order to facilitate design calculations as much as possible design curves ave been plotted for each of the various turbulent flow regimes. These are f the same general type as the familiar Moody curves, giving friction factor s a function of Reynolds Number, and are used in exactly the same way. The nly difference is in the roughness parameters. As discussed previously, it s incorrect to attempt to lump all roughness effects together in terms of any ind of equivalent dimension. Each flow regime requires both its own roughess parameter and friction characteristic curve.

Fig. 5 contains the curves for semi-smooth turbulent flow, with the lowerost curve being that for smooth flow. The curves are plots of equation (9), or parametric values of the roughness function ΣE defined in equation (10). alues of drag coefficient C_D (ordinarily assumed constant with sufficient ccuracy for design purposes, at usual values of N_R) to use in evaluating E or each repeating roughness element are tabulated in Figure 5-a. Use of hese curves of course requires that the roughness element height $\underline{h}$ be great nough to pierce the laminar boundary film; otherwise, smooth flow will preail. The thickness of this boundary layer is a decreasing function* of



FRICTION FACTOR FOR SEMI-SMOOTH TURBULENT FLOW

FIGURE 5

.e. $\frac{\delta}{D} = \frac{32.8}{N_R \sqrt{f_s}}$

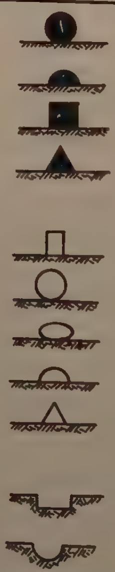
SEMI-SMOOTH FLOW EQUATION : $f = f_s\left[1 + 67.2\sum\frac{p}{P}\frac{h}{\lambda}C_D\right]$

f_s = SMOOTH FLOW FRICTION FACTOR AT GIVEN REYNOLDS NUMBER.

$\frac{p}{P}$ = PERIPHERAL ROUGHNESS RATIO $= \dfrac{\text{TOTAL PERIPHERAL LENGTH OF ROUGHNESS ELEMENTS}}{\text{CONDUIT WETTED PERIMETER}}$

$\frac{h}{\lambda}$ = ROUGHNESS INDEX $= \dfrac{\text{ROUGHNESS HEIGHT}}{\text{ROUGHNESS SPACING}}$

C_D = COEFFICIENT OF DRAG FOR ROUGHNESS ELEMENT FORM APPROXIMATELY AS IN FOLLOWING TABLE



SPOT ROUGHNESS	TYPE OF ELEMENT	C_D
SPHERE		0.5
HEMISPHERE		0.4
CUBE		1.5
CONE		0.5
STRIP ROUGHNESS		
RECTANGULAR		1.9
CIRCULAR		0.8
ELLIPTICAL		0.5
SEMI-CIRCULAR		0.6
TRIANGULAR		1.5
DEPRESSION ROUGHNESS		
RECTANGULAR SLOT		1.0
HEMISPHERICAL PIT		0.2

DATA FOR USE WITH SEMI-SMOOTH FLOW CURVES

FIGURE 5A

Reynolds Number. This function is plotted on Figure 5-b. As long as the roughness height is significantly smaller than δ, it can be assumed that smooth flow will prevail. For simplicity, and for design purposes, no attempt has been made to delineate a transition function from smooth to semi-smooth flow. For particular elements, this could in theory be done by allowing for varying values of $\underline{h}$ and C_D at low Reynolds Numbers, until the separation points on the crests of the elements become stabilized.

The quasi-smooth function is shown in Fig. 6, again with the lower-most curve being that for smooth flow. These curves are plots of equation (12), with the roughness parameter ΣX as defined therein. The ratio v_w/V may range from 3/8 to 6/8, perhaps averaging 2/3. The coefficient c_w is yet unknown but would perhaps range from about 1/2 to nearly unity. The term $(c_w v_w/V)^3$ would therefore lie within the range of, say, 0.01 to 0.40. In the absence of better information, a value of 0.10 would be probably reasonable and conservative. The value of 0.05 checks well with such observational data as are currently available for this type of flow.

Since the friction factor in hyper-turbulent flow depends on both the relative roughness spacing r_o/λ and the form of elements, a different set of curves would be required for each form of element. On Figures 7, 8, and 9 curves have been plotted for the most common types of rough surfaces producing this type of flow, as well as its limiting state of fully normal turbulent flow. These surfaces are those containing regular corrugations, sharp-edged circumferential strips, and densely-packed spot roughnesses, such as sand grains.

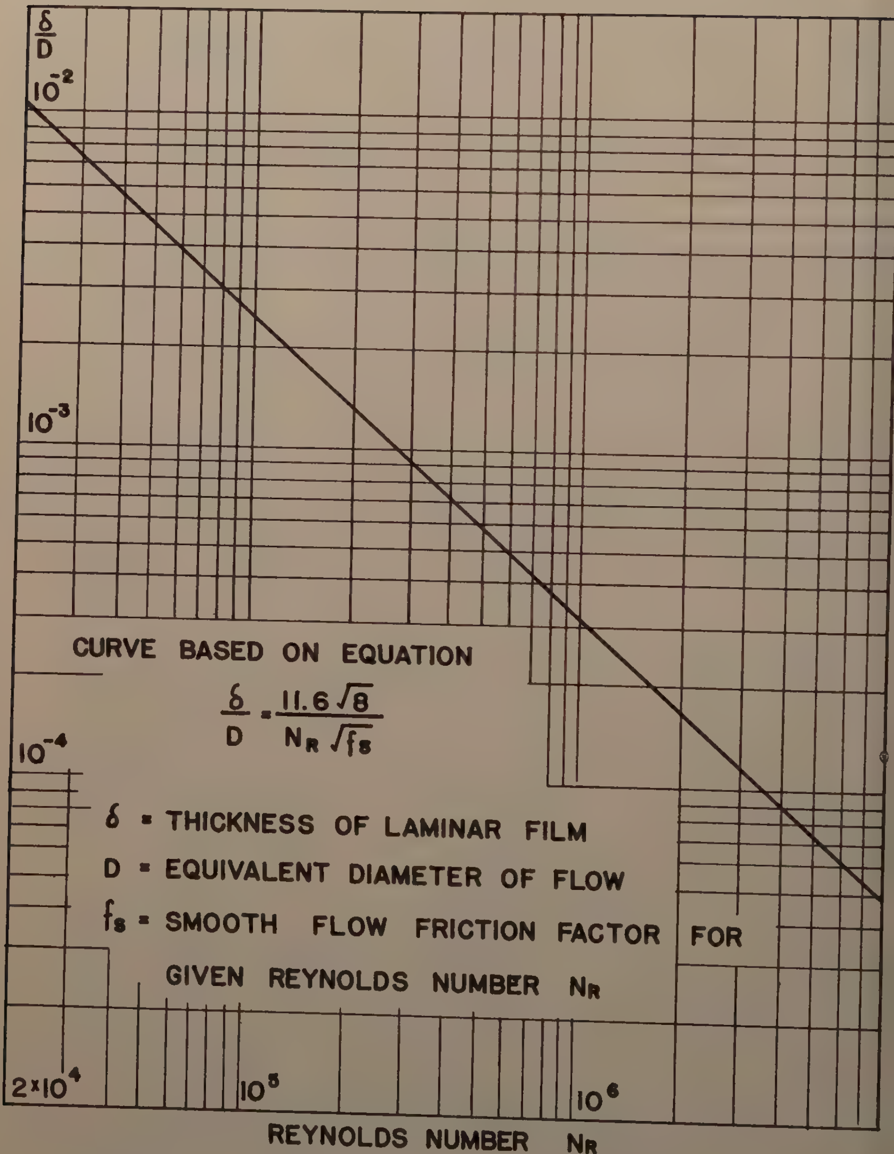
Each set of curves is plotted with the relative roughness spacing r_o/λ as parameter. If the effect of some other form of roughness is needed, a judicious extrapolation or interpolation of the results obtained from these curves (which have been shown(15) to correlate experimental results obtained from many different sources) might be employed. It may be noted too that the curves, since they are experimental curves, define the entire transition from smooth flow to normal turbulent flow. As noted before, this includes a segment of semi-smooth flow as well as the hyper-turbulent regime proper.

Discrimination between Flow Regimes

The curves of Figures 5 through 9 will permit the solution of any ordinary problem in conduit flow (in which elastic, surface tension and gravitational effects can be neglected), provided only that the roughness geometry and flow regime are known. However, the problem of discriminating between semi-smooth, quasi-smooth and hyper-turbulent flow has not yet been discussed.

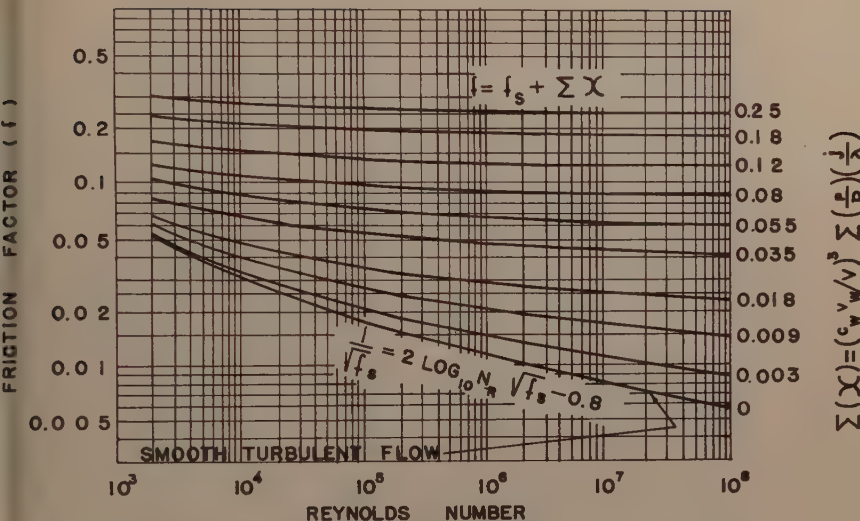
Quasi-smooth flow will occur over roughness depressions in which stable vortices can be maintained. This means that these depressions must be at least nearly as deep as they are long. Otherwise they will develop either semi-smooth or hyper-turbulent flow. There exists some evidence, not yet definitive, that at sufficiently high Reynolds Numbers normal turbulent flow may under some conditions change to quasi-smooth flow. This is indicated when a rising or horizontal f-N_R curve begins to drop. Until more information becomes available on this condition, it is conservative to ignore it in design predictions.

The problem of determining whether, for a given surface, semi-smooth or hyper-turbulent flow will occur is more difficult. For a given type of roughness element, the spacing of elements is the critical factor. The boundary



LAMINAR BOUNDARY FILM THICKNESS

FIGURE 5B



FRICTION FACTOR FOR QUASI-SMOOTH (SKIMMING) FLOW

FIGURE 8

between the two regimes, for a given Reynolds Number and conduit would be that spacing for which the friction factor attains its maximum value. For spacings exceeding this critical value, causing semi-smooth flow, equation (9) shows friction factor to decrease as the spacing increases. For closer spacing, producing hyper-turbulent or normal flow, equations (11) and (8) show the friction factor to decrease as the spacing decreases.

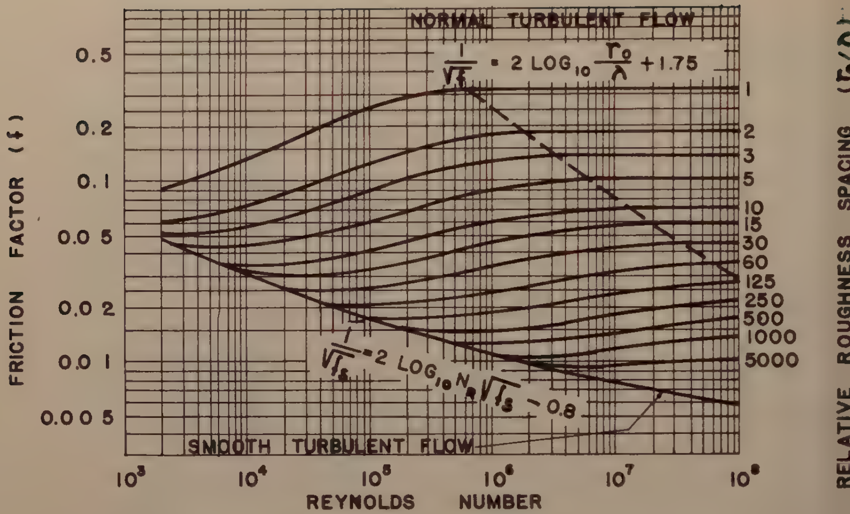
The critical spacing (or perhaps another factor) for a given type of element could therefore be determined by equating the friction factor expressions for semi-smooth and hyper-turbulent flow. Thus:

$$\frac{1}{\sqrt{f_s \; (1 + 67.2E)}} - 2\log_{10}\frac{r_o}{\lambda} = 1.75 + \phi \qquad (13)$$

Solution of equation (13) for the spacing λ, or whatever factor is unknown, determines the boundary between semi-smooth and hyper-turbulent flow for the given conditions. The presence of the function ϕ makes such solution quite difficult. As a reasonable approximation, assume $f_s = 0.01$ and $\phi = 0$. Then:

$$67.2\left(\frac{r_o}{\lambda}\right)\left(\frac{h}{r_o}\right)\left(\frac{p}{P}\right)\left(C_D\right) + 1 = \frac{100}{(1.75 + 2\log_{10}\frac{r_o}{\lambda})^2} \qquad (14)$$

Equation (14) is plotted on Figure 10, giving the critical relative roughness spacing r_o/λ as a function of the relative roughness height, with the factor (p/P) (C_D) parametric.



FRICTION FACTOR FOR HYPER-TURBULENT FLOW, CORRUGATION STRIP ROUGHNESSES.

FIGURE 7

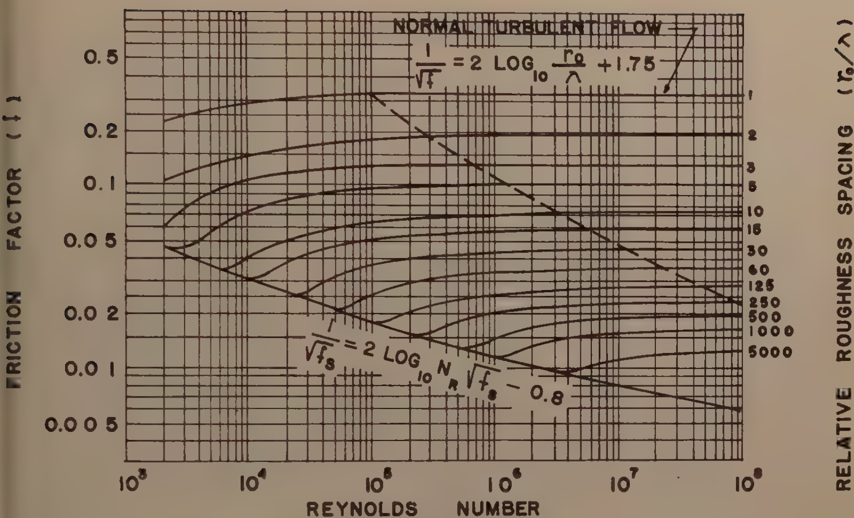
Applications in Practical Hydraulic Analysis

The application of these concepts and design curves to practical flow problems requires knowledge of the geometry of the surface roughness elements. In the case of surfaces of regular roughness pattern, such as corrugated metal or uniform sand or gravel, this geometry is known or easily determined. Such surfaces, if the roughness elements are closely spaced, nearly always produce hyper-turbulent flow, and at sufficiently high Reynolds Numbers, normal turbulent flow.

Materials commonly used for water and sewer pipe, and for gas and oil transmission - including concrete, cast iron, clay, welded steel, etc., - are, at least when new, essentially smooth, except for isolated roughness elements and for the joints. If rivets, bolts, etc., are present, their effects are easily computed by the semi-smooth flow equation. The joints may cause either a protuberance or a depression, and the effects may be calculated by the semi-smooth or quasi-smooth relations accordingly. Although the exact geometry of the elements often may not be known, it is believed that the designer could estimate these factors with at least as much confidence and accuracy as a roughness coefficient or equivalent sand roughness. Materials of this type will usually produce either semi-smooth or quasi-smooth flow, and the appropriate curves can be used accordingly.

Surfaces of variable or irregular roughness are of course more difficult to evaluate. However, the principles of the various turbulence regimes must still be valid, so that friction factors for such surfaces could still be estimated by judicious application and extension of these principles.

If, for example the surface consists of a combination of several types of isolated protuberances and depressions, the semi-smooth and quasi-smooth



FRICTION FACTOR FOR HYPER–TURBULENT FLOW, SHARP EDGED STRIP ROUGHNESS.

FIGURE 8

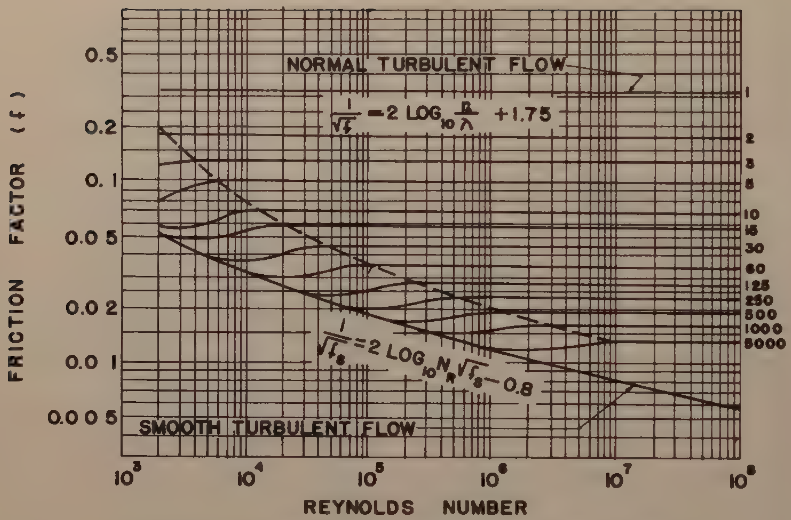
equations could be combined, as follows:

$$f = f_s \left[1 + \Sigma\, 67.2\, C_D \frac{p}{P} \left(\frac{h}{\lambda}\right) \right] + \Sigma \frac{p}{P} \left(\frac{j}{\lambda}\right) \left(\frac{c_w v_w}{V}\right)^3 \qquad (15)$$

If the elements are close enough together to produce hyper-turbulent or normal flow, the hyper-turbulence equation must be modified to allow for elements of various forms and spacings. If only the spacing is variable, its average value may be used with sufficient accuracy, since it is basically the number of turbulence sources that determines the wall zone turbulence patterns. If both the spacing and roughness form are variable, the average value of spacing may again be used in the hyper-turbulent flow equation, but judgment must be exercised as to which form of transition function ϕ should be used, depending upon the prevalent form in the roughness pattern.

When occasional elements project much farther into the flow than others, the effect is that of superimposing isolated wakes and drag forces on the typical hyper-turbulent flow near the smaller elements. It is reasonable in this situation to compute the friction factor due to the hyper-turbulent flow near the ordinary elements, and then add increments of friction factor corresponding to the larger elements. The resulting relation would be:

$$f = \frac{1}{\left(2\log_{10} \frac{r_o}{\lambda} + 1.75 + \phi\right)^2} + 2\Sigma \frac{p}{P} \left(C_D\right) \left(\frac{h}{\lambda}\right) \qquad (16)$$



FRICTION FACTOR FOR HYPER-TURBULENT FLOW
UNIFORM SPOT ROUGHNESSES.

FIGURE 9

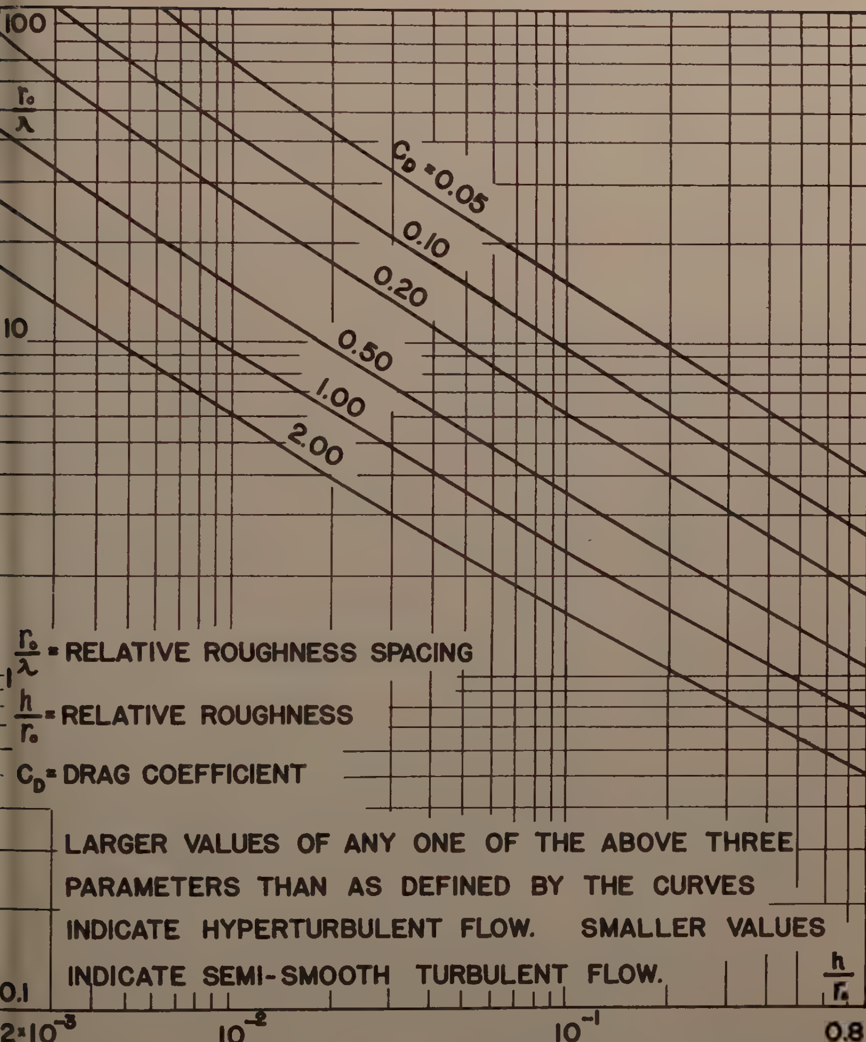
the second term in equation (16) is an approximation based on the assumption that the velocity near the wall which impinges on the projecting element is about 0.7 of the average velocity in the cross-section. The element height $\underline{h}$ must be measured from the crests of the smaller adjacent elements.

In the case of a conduit with part of its perimeter very rough and part smooth (such as a corrugated pipe with paved invert, or a pipe with longitudinal rows of closely-spaced rivet projections) it is obvious that the over-all friction factor will be intermediate between that for a smooth flow and that for a hyper-turbulent flow corresponding to the rough section of the surface. Most of the energy expenditure due to friction in a flow occurs in the process of vortex generation at the conduit surface. Thus it is reasonable to take the proportion of rough surface area to smooth surface area as the basis for determining the bulk friction factor. The resulting equation is:

$$f = \frac{p_s}{P}\left(f_s\right) + \frac{p}{P}\left(\frac{1}{\left(2\log_{10}\frac{r_o}{\lambda} + 1.75 + \phi\right)^2}\right) \qquad (17$$

where p_s and p are the smooth and rough segments of the periphery, respectively.

In the relatively rare case of a surface composed of "random" roughness elements, of various shapes, sizes, and spacings, the tendency is for the friction factor - Reynolds Number curve to be horizontal throughout the entire turbulent range. That is, such a statistically random roughness distribution produces a statistically random vorticity pattern near the wall, and therefore normal turbulent flow, even at low Reynolds Numbers. The friction factor for such a surface can probably be estimated by the normal turbulent flow equation, using the average spacing of predominating larger elements on the surface.



BOUNDARY FUNCTION FOR DISCRIMINATION BETWEEN SEMI-SMOOTH AND HYPER-TURBULENT FLOW

FIGURE 10

Reliability and Utility in Hydraulic Design

The concepts, equations, and curves presented in this paper provide a design and analysis tool for flow problems which is substantially retional in nature, as compared with the largely empirical methods in common use. It is somewhat more difficult to use than previous methods, a defect inherent in any attempt to place the complexities of hydraulic phenomena on a more rational design basis. However, this difficulty is materially alleviated through use of the design curves included herein.

The slightly increased effort required of the designer is, moreover, more than justified if his work can thereby be placed on a sounder scientific footing than is possible near present procedures, which necessarily involve an important element of uncertainty in the choice of a "roughness coefficient," "equivalent sand diameter" or other roughness measure, as well as uncertainty in the variability of this measure.

The advantages and potentialities of the concepts of this paper, for solution of practical problems in both open and closed conduit flow, would appear to justify intensive experimental study for verification purposes. It has already been shown(16) that the concepts correlate data obtained from many different sources, on many types of roughnesses, in both open and pressure channels. It is believed that this is also substantially true for more recent experimental work. However, further study, directed to the specific end of elucidating and refining the concepts and design methods based on them is very desirable.

Until such further studies become available, it is believed that judicious use of the equations and curves presented herein will suffice even at present to give design results at least as reliable, and often more reliable, than any other method in current use.

ACKNOWLEDGMENT

The figures for the paper were prepared by A. S. Winston and M. R. Jurkovich, senior students at V.P.I. Computations for several of the design curves were made by M. A. Person, while a graduate student at V.P.I.

GLOSSARY

A = a constant in the smooth flow velocity distribution equation, taken as 5.5

a = cross-sectional area of flow

B = a constant in the hyper-turbulent flow velocity distribution equation, taken as 8.5 for axial and 7.8 for two-dimensional flows.

C_D = discharge coefficient for surface roughness element.

c_w = a coefficient less than unity such that $c_w v_w$ is the peripheral velocity of a stable depression vortex

D = equivalent diameter of conduit $= 4 \frac{a}{P}$

E = roughness element function $= C_D \frac{p}{P} \frac{h}{\lambda}$

f = Darcy friction factor

f_s = friction factor in smooth turbulent flow

H_f = head loss to uniform surface friction $= f \frac{L}{D} \frac{V^2}{2g}$

j = longitudinal dimension of surface depression

h = height of roughness element

k = von Kàrman turbulence constant, taken as 0.40

L = length of reach in conduit

N_F = Froude Number

N_R = Reynolds Number (used as $\frac{DV}{\gamma}$ or $\frac{4aV}{\gamma\ P}$

N_m = Mach Number

N_W = Weber Number

P = wetted perimeter of conduit

p = total peripheral length of roughness elements at a section

p_s = peripheral length of smooth wall surface

r_o = equivalent radius of conduit

r = radius of rounding of roughness element

s = transverse spacing of roughness elements

V = average velocity of flow in the cross-section

v = velocity at a distance y from the conduit surface

v_c = maximum velocity in the cross-section

v_s = shear velocity, whose square is the ratio of wall shearing-stress to fluid density

v_w = velocity (turbulent) close to the wall

y = distance coordinate from wall to point in flow cross-section

δ = thickness of laminar boundary layer

χ = depression vortex function in quasi-smooth flow, $\left(\frac{c_w v_w}{V}\right)^3 \ \frac{j}{\lambda}\ \frac{p}{P}$

ϕ = additive element function in hyper-turbulent flow

λ = center-line longitudinal spacing of roughness elements

ν = kinematic viscosity of fluid

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GROUND WATER FLOW IN THE NETHERLANDS COASTAL DUNES

David K. Todd,[1] A. M. ASCE, and L. Huisman[2]

SYNOPSIS

A mathematical analysis of the ground water conditions in that portion of .e Netherlands coastal dunes which furnish a water supply to Amsterdam is :esented. The problem is treated two-dimensionally with a fresh-water lens oove and a fresh-water pocket below a clay layer under the dunes. Salt water nderlies the entire area. Beginning with the basic equations describing the :ound water situation, assumptions and boundary conditions necessary to ob-in solutions are described. Results give ground water movements and oundaries of the fresh-water lens and pocket. The method of solution is oplicable to related situations elsewhere.

INTRODUCTION

The principal source of water supply for the City of Amsterdam, etherlands, is a portion of the dunes along the North Sea coast extending uthward from Zandvoort for a length of 10 kilometers. Beginning in 1853 ater was collected from a series of shallow drainage ditches; subsequent de-lopments expanded the ditch system and added deep wells.

Fig. 1 shows a cross-section of the area perpendicular to the coast. The nes parallel the coast, are about 4 kilometers wide, and are mostly 5 to 10 eters above mean sea level with maximum elevations of 30 meters. The erage height of the ground water table is only two meters above mean sea vel with the highest level at about six meters. The dunes are bounded on the est by the North Sea and on the east by a 4-kilometer wide strip of flat land th a ground water table varying from +1.0 to -0.6 meter mean sea level. ast of this strip lies the Haarlem Lake Polder with a ground water level 5 eters below mean sea level.

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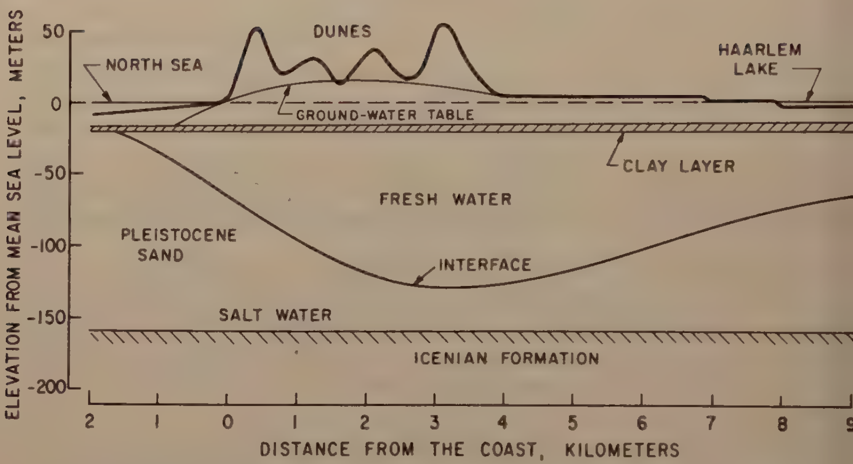

FIG. 1 CROSS-SECTION OF DUNES PERPENDICULAR TO THE COAST SHOWING WATER DISTRIBUTION IN 1850 (AFTER BIEMOND (1))

A large number of wells and test holes have enabled the geologic structur of the dunes to be defined. In terms of ground water movement four strata c be distinguished. The dunes are an aeolian formation and from ground surfa to 13 meters below mean sea level consist of a uniform fine Holocene (Recer sand. A much less pervious stratum of clay and peat occupies a layer from 13 to 20 meters below mean sea level. Below this coarse Pleistocene sand extends to 160 meters below mean sea level. The final stratum, extending downward from 160 meters below mean sea level, is an Icenian (Pliocene) formation of very fine and silty sand which may be considered the lower boundary of the dune aquifers.

The dunes are a unique source of fresh ground water in the Netherlands a most of the country is underlain by salt water. Precipitation infiltrates into the dunes to form a lens of fresh-water above the clay layer, while a portion of this water percolates through the clay layer to form a fresh-water pocket floating on salt water. The distribution of fresh and salt waters in 1850, be- fore withdrawals for water supply were initiated, is indicated in Fig. 1.

A considerable body of literature relating to sea water intrusion and fres salt water relationships exists.(6) In particular, contributions by Dutch engi neers have been most significant toward understanding these complex ground water hydrology problems.(4) Lindenbergh(5) has described the occurrence and development of ground water from that portion of the dunes supplying water to the City of Leyden, Netherlands. For the dune area supplying water to Amsterdam, Biemond (1) has published an excellent account of the ground water development, its history and problems. Detailed investigations by Edelman (2) have dealt with evaluating the ground water distribution in the dunes in relation to planned ground water withdrawals. A paper by Huisman describes methods for determining permeability coefficients for the various dune strata. These coefficients have been adopted for the analysis presente here and are included in the notation on the next page.

The following mathematical analysis enables the ground water movement and the boundaries of the fresh-water lens and pocket to be determined. As- sumptions and boundary conditions to obtain solutions have been adopted to f the dune area as of 1850. At that time the Haarlem Lake had not been re- claimed, and its elevation was about mean sea level; therefore, no movemen

f salt water occurred below the fresh water. Today, with the Haarlem Lake eclaimed as a polder, the ground water level lies below mean sea level so nat a landward flow of salt water exists. This introduces mathematical omplications which can only be solved with the aid of electronic computers.

Results presented below are based upon data for the area under study. nformation on dimensions of the dunes, geologic strata, and permeabilities as available from exploratory measurements and previous studies. Although ne quantitative results necessarily are not generally applicable, the method f analysis is. Any coastal ground water situation involving a dynamic equili- rium of fresh water over a stationary salt water body can be treated in a imilar manner. In fact, with only minor modifications for radial flow, the rocedure can be applied to analyze ground water conditions on an unconfined ceanic island.

The authors have tried to emphasize the analytic procedure involved by arrying a specific problem through to a quantitative end. It is hoped that the rocedure will prove helpful to engineers who may encounter similar or relat- d ground water situations.

Notation

= cross-sectional area of the fresh-water lens above O.D., m^2

$_1$, C_2, C_3, C_4, C_5, C_6 = integration constants

= average vertical thickness of the fresh-water lens, m

.h = fresh-water depth in excess of h_s landward of the lens, m

s = depth of salt water above the clay layer, m

c = permeability of the clay layer = 0.002 m/day

L = permeability of the Holocene sand formation, m/day

P = permeability of the Pleistocene sand formation = 32 m/day

= half-width of the fresh-water lens perpendicular to the coast = 3000 m

= half-width of the fresh-water pocket including tongues, m

= rate of effective precipitation, mm/year or m^3/day

.D. = Amsterdam ordnance datum = +0.15 m mean sea level

= horizontal fresh-water flow in the fresh-water lens, m^3/day

= fresh-water flow in the pocket, m^3/day

Q = pumpage rate from one side of the pocket at $x = \ell$, $y = y_c$

Q_ℓ = half of the fresh-water flow from the lens into the pocket, m^3/day

Q_t = fresh-water flow into a tongue, m^3/day

x = horizontal coordinate measured from the center of the dunes perpendicular to the coast, m

y = fresh-water depth in the pocket, m

y_c = value of y at the edge of the fresh-water lens, m

y_m = maximum value of y at the center of the fresh-water pocket, m

z = fresh-water height in the lens measured from O.D., m

z_a = average value of z, m

α = constant

β = constant

γ = constant = $10^{12}\beta$

λ = constant = $\sqrt{\frac{7K_L D}{K_c}}$

ρf = density of fresh-water = 1.000 gm/cm^3

ρs = density of salt water = 1.020 gm/cm^3

ϕ = piezometric level (or potential) of the fresh-water pocket, m

ϕa = average value of ϕ, m

ψ = piezometric level (or potential) of the salt water = -0.15 m

Assumptions

For purposes of analysis the definitive sketch in Fig. 2 will serve as a reference. Although idealized, the sketch is a good approximation to the actual dunes cross-section before reclamation of the Haarlem Lake. The problem will be treated in two dimensions with flows referring to a unit thickness of one meter. The fresh-water lens lies above the clay layer, while the fresh-water pocket with its lateral tongues lies below. All interfaces are boundaries of negligible thickness. Because the salt water is at rest, the piezometric level ψ for the salt water may be assumed constant at mean sea level, or -0.15 m O.D. (Amsterdam ordnance datum). Salt water occupies the section above the clay layer on the seaward (left) side of the lens; however, on the landward (right) side there is fresh water. The Ghyben-Herzberg relationship for hydrostatic equilibrium of the two fluids can be expressed as

$$\rho_s h_s = \rho_f h_s + \rho_f \Delta h \qquad (1)$$

vhere ρ_f and ρ_s are the fresh and salt water densities respectively, h_s is the ;alt water depth above the clay layer, and Δh is the fresh-water depth greater han h_s. Solving for Δh and inserting values gives

$$\Delta h = \frac{\rho_s - \rho_f}{\rho_f} h_s = \frac{1.020 - 1.000}{1.000} (13) = 0.26 \text{ m} \tag{2}$$

Therefore, a constant ψ gives a fresh-water elevation of -0.15 + 0.26 = + 0.11 n O.D. on the landward side of the lens as shown in Fig. 2.

Analysis of the Tongue

The tongues are those portions of the fresh-water pocket extending beyond he fresh-water lens. Because the salt water potential is constant, the tongues on the landward and seaward side of the lens are symmetrical. Flow within a ongue consists of horizontal movement from the central portion of the pocket nd an upward vertical movement through the clay layer.

From the notation and Fig. 2, the tongue analysis can be based on the Darcy quation for flow within the tongue,

$$Q = -K_p y \frac{d\phi}{dx} \tag{3}$$

nd for upward flow through the clay layer,

$$\frac{dQ}{dx} = -\frac{K_c}{7}(\phi - 0.11) \tag{4}$$

t any point on the fresh-salt water interface the potentials in the two water odies must be in equilibrium; therefore,

$$(\phi + y + 20)1.000 = (\psi + y + 20)1.020 \tag{5}$$

3ut the salt water potential ψ is a constant equal to -0.15 m, hence Eq. (5) educes to

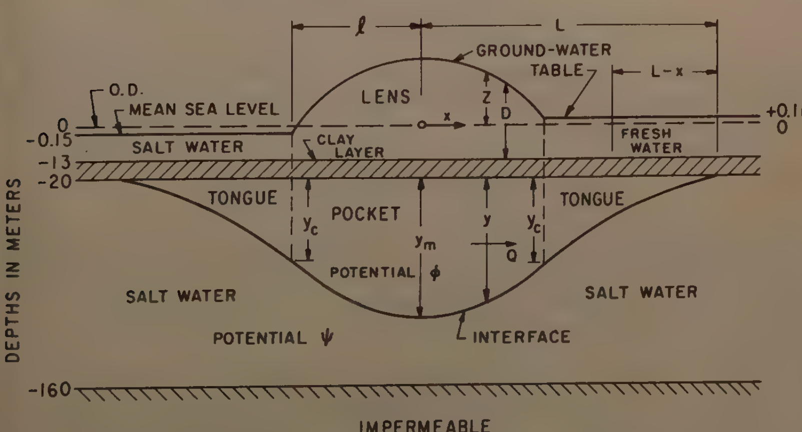

FIG. 2 DEFINITIVE SKETCH OF DUNES CROSS-SECTION

$$\phi = 0.02y + 0.25 \tag{6}$$

Combining Eqs. (3), (4), and (6) yields the tongue flow equation

$$Q^2 = \frac{0.0008 K_p K_c}{7}\left(\frac{y^3}{3} + 3.5y^2\right) + C_1 \tag{7}$$

where C_1 is an integration constant. From the boundary condition at the end of the tongue that y = 0 and Q = 0, $C_1 = 0$. Hence Eq. (7) reduces to

$$Q = \sqrt{\frac{0.0008 K_p K_c}{7}\left(\frac{y^3}{3} + 3.5y^2\right)} \tag{8}$$

Now from Eqs. (3) and (6)

$$Q = -0.02 K_p y \frac{dy}{dx} \tag{9}$$

so that equating Eqs. (8) and (9)

$$dx = -5\sqrt{0.42\frac{K_p}{K_c}}\,\frac{dy}{\sqrt{y+10.5}} \tag{10}$$

which upon integration yields

$$x = -\sqrt{\frac{42 K_p}{K_c}}\sqrt{y+10.5} + C_2 \tag{11}$$

where C_2 is an integration constant. Again at the end of the tongue, x = L an y = 0, so that

$$C_2 = L + \sqrt{\frac{441 K_p}{K_c}}$$

and Eq. (11) becomes

$$L - x = \sqrt{\frac{7K_p}{K_c}}\left[\sqrt{6y+63} - \sqrt{63}\right] \tag{12}$$

Eq. (12) defines the interfacial boundary of the tongue. The length of the tongue $L - \ell$ can be given by the boundary condition $x = \ell$, $y = y_c$. Substitutin in Eq. (12) gives the length

$$L - \ell = \sqrt{\frac{7K_p}{K_c}}\left[\sqrt{6y_c+63} - \sqrt{63}\right] \tag{13}$$

With the permeabilities known, the length of the tongue can be found if y_c is known. This value will be obtained subsequently in the analysis.

The form of the tongue interface can be obtained directly from Eq. (12) b inserting values for K_c and K_p; thus,

$$6y + 63 = \left[\frac{L-x}{334.66} + 7.937\right]^2 \tag{14}$$

Substituting various values of L-x gives the coordinates of the interface list on Table 1. Coordinates of the piezometric surface for the tongue, obtained from Eq. (6), are also listed.

Table 1 - Coordinates of the Tongue Interface and Piezometric Surface

L-x, m	y, m	ϕ, m
0	0	0.25
1000	9.4	0.44
2000	21.8	0.69
3000	37.1	0.99
4000	55.4	1.36
5000	76.7	1.78
6000	101.0	2.27
7000	128.3	2.82

Analysis of the Fresh-Water Lens and Pocket

The fresh-water lens and pocket are symmetrical about the center line of ne dunes as shown in Fig. 2. Water is supplied to the lens by vertical percotion from effective precipitation. This equals gross precipitation minus vapotranspiration and amounts to a rate N, assumed constant. Water leaves ne lens either by outward horizontal flow at the edge of the lens or by vertial downward flow through the clay layer.

In Fig. 3 is shown a sketch of half of the fresh-water lens cross-section. he downward flow through the clay layer in the segment dx can be given by

$$dQ = \frac{K_c(z-\phi)}{7}\,dx \tag{15}$$

imilarly, the continuity equation for the same segment is

$$q + N\,dx - (q + dq) - \frac{K_c(z-\phi)}{7}\,dx = 0 \tag{16}$$

r

$$\frac{dq}{dx} = N - \frac{K_c(z-\phi)}{7} \tag{17}$$

here q is the horizontal fresh-water flow in the lens. The Darcy equation r this flow is

$$q = -K_L D \frac{dz}{dx} \tag{18}$$

here D is the average thickness of the lens.

Although D is a function of z, as a good approximation it may be assumed onstant and given by (from (3))

$$K_L D = 100 + 12\,z_a \tag{19}$$

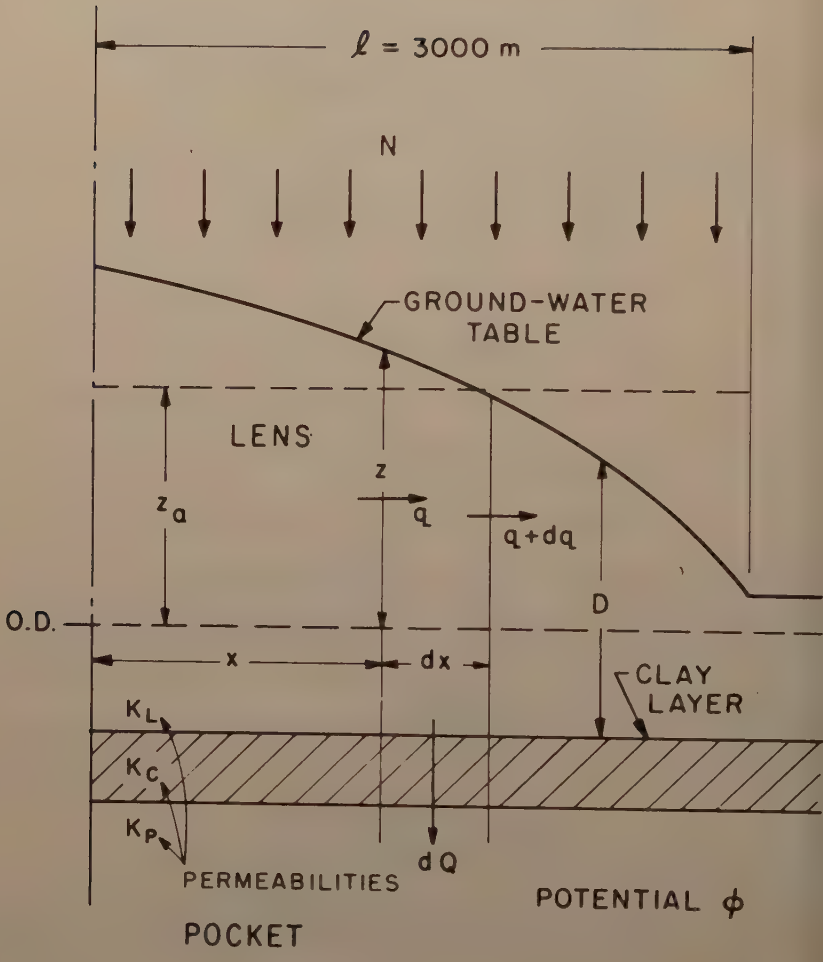

FIG. 3 DEFINITIVE SKETCH OF HALF OF FRESH-WATER LENS CROSS-SECTION

here z_a is the average z defined by

$$z_a = \frac{1}{\ell}\int_0^{\ell} z\,dx \tag{20}$$

ombining Eqs. (17) and (18)

$$\frac{dq}{dx} = -K_L D\frac{d^2z}{dx^2} = N - \frac{K_c(z-\phi)}{7} \tag{21}$$

r

$$\frac{d^2z}{dx^2} - \frac{K_c z}{7K_L D} + \frac{N}{K_L D} + \frac{K_c\phi}{7K_L D} = 0 \tag{22}$$

or simplicity let $\lambda = \sqrt{\dfrac{7\,K_L\,D}{K_c}}$, then Eq. (22) becomes

$$\frac{d^2z}{dx^2} - \frac{z}{\lambda^2} + \frac{\frac{7N}{K_c}+\phi}{\lambda^2} = 0 \tag{23}$$

his differential equation can be solved when

$$\frac{d^2\phi}{dx^2} = 0$$

wo possibilities are ϕ = constant and $\phi = ax + b$. As ϕ varies only a small mount over relatively long horizontal distances (see Table 1), as a simple pproximation the average

$$\phi_a = \frac{1}{\ell}\int_0^{\ell}\phi\,dx \tag{24}$$

ill suffice. Substituting in Eq. (23)

$$\frac{d^2z}{dx^2} - \frac{z}{\lambda^2} + \frac{\frac{7N}{K_c}+\phi_a}{\lambda^2} = 0 \tag{25}$$

hich has as its solution

$$z = C_3 e^{-\frac{x}{\lambda}} + C_4 e^{+\frac{x}{\lambda}} + \frac{7N}{K_c} + \phi_a \tag{26}$$

here C_3 and C_4 are integration constants.

From the boundary condition that

$$\frac{dz}{dx} = 0 \quad \text{when} \quad x = 0$$

follows that $C_3 = C_4$. Hence Eq. (26) becomes

$$z = 2C_3\cosh\frac{x}{\lambda} + \frac{7N}{K_c} + \phi_a \tag{27}$$

With this function of z, it is not possible to solve for $y = f(x)$. Instead, it is ecessary to find a good approximation of $z = f(x)$ which will lead to a solution r $y = f(x)$. Assume a relationship of the form

$$z = \phi + \alpha + \beta x^4 \tag{28}$$

Combining with Eq. (15) gives

$$z - \phi = \alpha + \beta x^4 = \frac{7}{K_c}\frac{dQ}{dx} \tag{29}$$

or, upon integrating

$$\frac{7}{K_c} Q = \alpha x + \frac{\beta}{5} x^5 + C_5 \tag{30}$$

At the center of the lens x = 0 and Q = 0, so that the integration constant $C_5 = 0$; hence

$$\frac{7}{K_c} Q = \alpha x + \frac{\beta}{5} x^5 \tag{31}$$

The variable Q can be replaced by y upon substituting in Eqs. (3) and (6), leading to

$$\alpha x + \frac{\beta}{5} x^5 = -\frac{0.14 K_p}{K_c} y \frac{dy}{dx} \tag{32}$$

or, after integrating

$$\frac{\alpha}{2} x^2 + \frac{\beta}{30} x^6 + C_6 = -\frac{0.07 K_p}{K_c} y^2 \tag{33}$$

Eq. (33) relating x and y defines the shape of the fresh-water pocket interface under the lens. Its solution depends upon finding the constants α, β, and C_6. Boundary conditions include

(a) $x = 0$ at $y = y_m$
(b) $x = 3000$ at $z = +0.11$
(c) $x = 3000$ at $y = y_c$

From the first condition for assumed values of y_m, C_6 can be evaluated. Four cases, involving different values of y_m, will be considered and are listed in Table 2 with the corresponding C_6 values.

Table 2

Case	x	y_m, m	$10^{-6} C_6$
1	0	50	- 2.8
2	0	75	- 6.3
3	0	100	-11.2
4	0	150	-25.2

Substituting for ϕ from Eq. (6) in Eq. (28) and inserting the second and third boundary conditions above yields

$$0.11 = 0.02 y_c + 0.25 + \alpha + (3000)^4 \beta \tag{34}$$

Letting $\gamma = 10^{12}\beta$, this becomes

$$\alpha + 81\gamma = -0.14 - 0.02 y_c \tag{35}$$

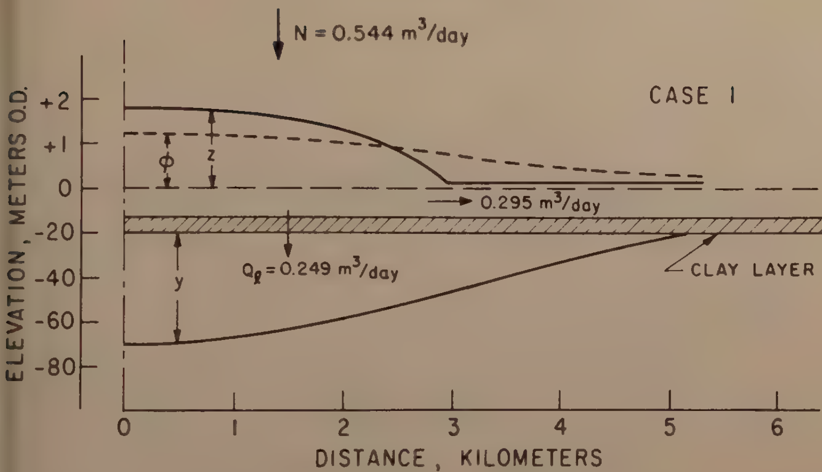

FIG. 4 FRESH-WATER DISTRIBUTION, CASE I

ubstituting values for x and y from the third boundary condition into Eq. (33),

$$\frac{(3000)^2}{2}\alpha + \frac{(3000)^6}{30}\beta + C_6 = -1120 y_c^2 \tag{36}$$

hich reduces to

$$4.5\alpha + 24.3\gamma = -10^{-6} C_6 - 0.00112 y_c^2 \tag{37}$$

The total flow from the lens into the pocket amounts to $2Q_\ell$. One half of his quantity, or Q_ℓ, flows through the clay layer between the center and the dge of the lens. Thus, from Eq. (31) with $x = \ell = 3000$ meters and $_c$ = 0.002 m/day

$$3500 Q_\ell = 3000\alpha + \frac{(3000)^5}{5}\beta \tag{38}$$

r

$$Q_\ell = \frac{3\alpha + 48.6\gamma}{3.5} \tag{39}$$

ut from continuity

$$Q_\ell = Q_t \tag{40}$$

here Q_t is the fresh-water flow through the section y_c into the tongue. From q. (8), $Q = Q_t$ when $y = y_c$; hence

$$Q_t = \sqrt{\frac{0.0008 K_p K_c}{7}\left(\frac{y_c^3}{3} + 3.5 y_c^2\right)} \tag{41}$$

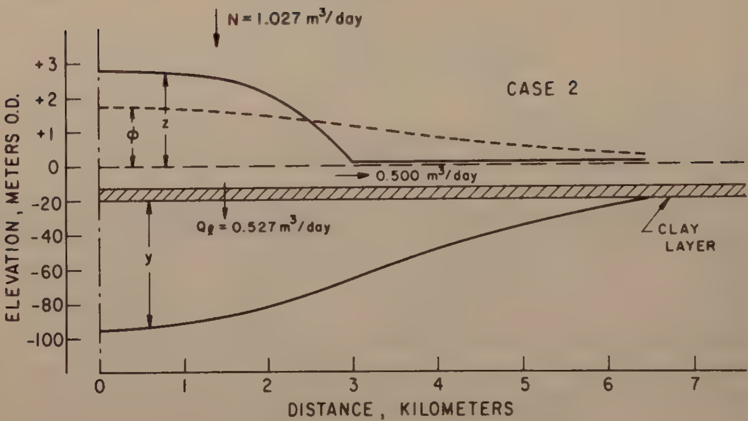

FIG. 5 FRESH-WATER DISTRIBUTION, CASE 2

Substituting values of Q_ℓ and Q_t from Eqs. (39) and (41), respectively, into Eq. (40) yields

$$\frac{3\alpha + 48.6\gamma}{3.5} = \sqrt{\frac{0.0008 K_p K_c}{7}\left(\frac{y_c^3}{3} + 3.5 y_c^2\right)} \qquad (4$$

Eqs. (35), (37), and (42) contain the unknowns α, γ, y_c, and C_6. The constant C_6 can be eliminated by assuming a value for y_m; the four cases of Table 2 will be considered. This reduces the problem to the solution of thre equations containing three unknowns. With these solved the functions y = f(x and z = f(x) can be obtained directly from Eqs. (6), (28), and (33). Results fo the four cases are summarized in Table 3 and Figs. 4-7.

Table 3 - Values for α, β, and y_c

Case	α	$\beta = 10^{-12}\gamma$	y_c, m
1	0.5298	-0.01476×10^{-12}	26.30
2	1.0295	-0.02560×10^{-12}	45.21
3	1.6445	-0.03807×10^{-12}	64.95
4	3.1681	-0.06700×10^{-12}	105.95

Comparison of Z-Curves

With z = f(x) now determined from the assumed Eq. (28), it is possible to compare this result with the more exact function according to Eq. (27). The two unknowns C_3 and N require two boundary conditions. The first of these is that z = + 0.11 at x = 3000, so that from Eq. (27)

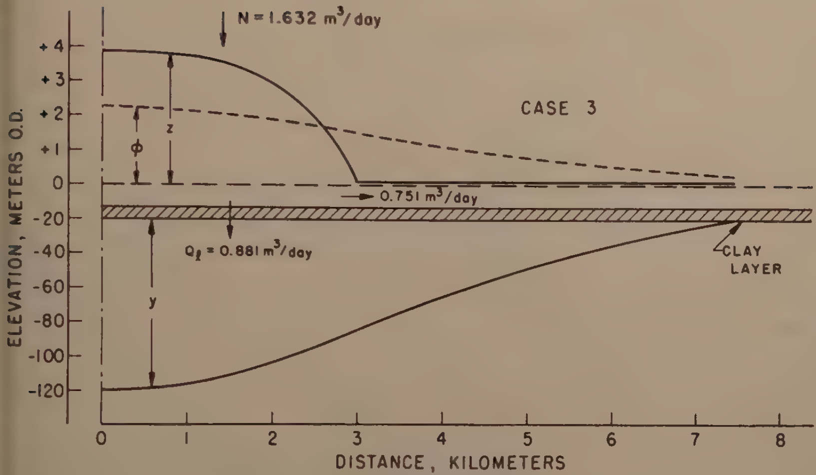

FIG. 6 FRESH-WATER DISTRIBUTION, CASE 3

$$0.11 = 2C_3 \cosh \frac{3000}{\lambda} + \frac{7N}{K_c} + \phi_a \tag{43}$$

ne second condition is that

$$\left[\int_0^{\ell} z\,dx\right]_{\text{Eqn.27}} = \left[\int_0^{\ell} z\,dx\right]_{\text{Eqn.28}} = \frac{A}{2} \tag{44}$$

ere A equals the cross-sectional area of the lens above O.D. Although the egral from Eq. (28) cannot be obtained directly because $\phi = f(y)$ leads to an iptic integral, it can be evaluated by planimetry. Thus, integrating and re-ranging Eq. (28) gives

$$\int_0^{\ell} \phi\,dx = \int_0^{\ell} z\,dx - \left(\ell\alpha + \frac{\ell^5}{5}\beta\right) \tag{45}$$

which all terms can be obtained. Integrating Eq. (27)

$$\left[\int_0^{\ell} z\,dx\right]_{\text{Eqn.27}} = \int_0^{\ell} \left[2C_3 \cosh \frac{x}{\lambda} + \frac{7N}{K_c} + \phi_a\right] dx \tag{46}$$

$$\frac{A}{2} = 2\lambda C_3 \sinh \frac{3000}{\lambda} + \left(\frac{7N}{K_c} + \phi_a\right) 3000 \tag{47}$$

s. (43) and (47) can now be solved for the unknowns C_3 and N. The quantity appearing in both equations can be found from Eq. (45), as its left side ials $\ell\phi_a$ (see Eq. (24)). The same is true for z_a in Eq. (19) as a part of λ. The resulting z-curves from Eqs. (27) and (28) are plotted in Fig. 8. A ximum difference of 14 per cent occurs along the Case 1 curve; the average ference for the four cases is 3 per cent. The good agreement substantiates assumed relationship of Eq. (28).

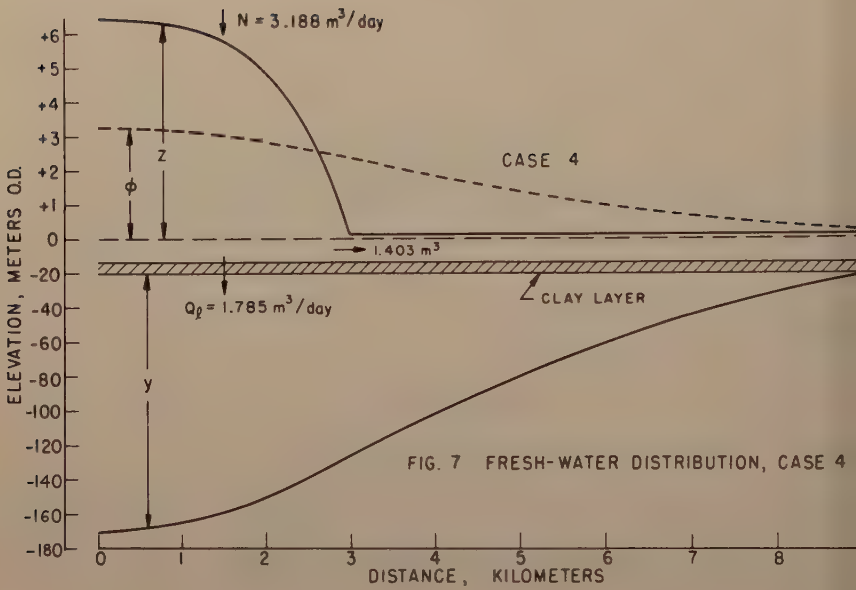

FIG. 7 FRESH-WATER DISTRIBUTION, CASE 4

Values of N and Q

Values of N for each of the four cases were obtained in the above analysi of z. Each N indicates the average recharge rate from precipitation minus evapotranspiration necessary to maintain the lens and pocket configurations In Table 4 are listed values of N as a precipitation rate in mm/yr and as a charge rate over one-half of the lens for an area of unit width in m^3/day.

Of the recharged water a quantity Q_ℓ percolates downward through the c layer on each half of the lens, while the remainder $N-Q_\ell$ flows outward at each edge of the lens above the clay layer. Values of Q_ℓ, found from Eq. (3 are listed in Table 4 together with $N-Q_\ell$ quantities.

Values of N, Q_ℓ, and $N-Q_\ell$ for the four cases are also shown on Figs. 4

Table 4 - Values of N and Q_ℓ

Case	N, mm/yr	N, m^3/day	Q_ℓ, m^3/day	$N-Q_\ell$, m^3/da
1	66.2	0.544	0.249	0.295
2	125.0	1.027	0.527	0.500
3	198.5	1.632	0.881	0.751
4	387.9	3.188	1.785	1.403

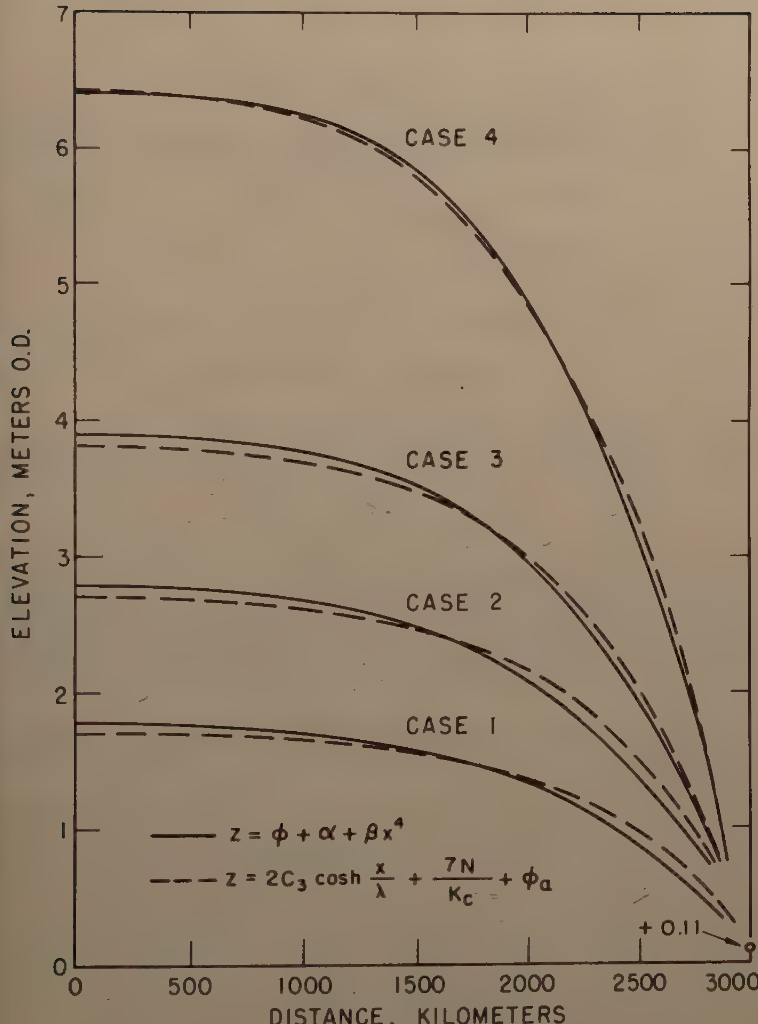

FIG. 8 COMPARISON OF Z-CURVES FOR SHAPE OF FRESH-WATER LENS

Pumpage from the Fresh-Water Pocket

If any water is extracted from the fresh-water pocket, the position of its terface will of course change. Under conditions in Netherlands water is umped at the edges of the lens (dunes) to minimize flow of water into the ngues where it is lost from a water supply standpoint. The position of the ocket interface can be determined for various pumpage rates at $x = \ell$.

Let ΔQ equal the amount of water pumped from one side of the pocket at $= \ell$, $y = y_c$. Eq. (40) may then be rewritten

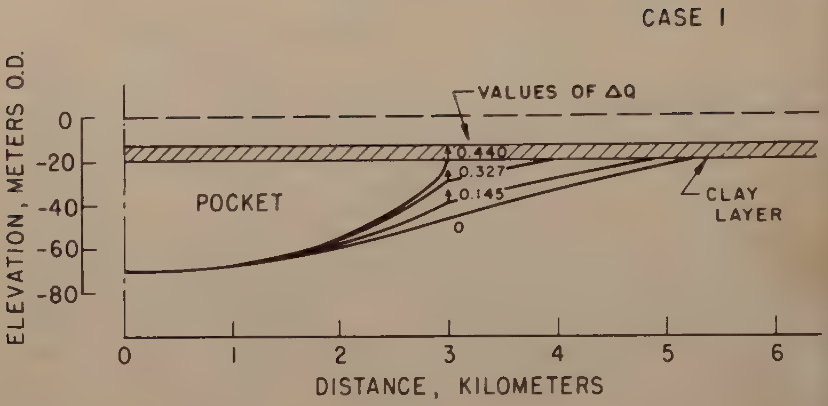

FIG. 9 FRESH-WATER DISTRIBUTION WITH PUMPAGE FROM THE POCKET CASE I

$$Q_\ell - Q_t = \Delta Q \tag{48}$$

so that ΔQ may be computed from Eqs. (39) and (41) for a given value of y_c in one of the four previous cases. The value of Q_ℓ in Eq. (39) depends upon α and γ, which can be obtained by solving Eqs. (35) and (37) simultaneously. Knowing α, β, and C_6, the shape of the pocket interface can be calculated from Eq. (33). Values of ΔQ for selected values of y_c in the four cases are listed in Table 5.

The length of the tongue with pumping is given, as before, by knowing y_c and substituting in Eq. (13). The shape of the tongue interface is given by Eq. (14) or Table 1.

Positions of the pocket and tongue interfaces for values of y_c in Table 5 are plotted in Figs. 9-12. It can be observed that as the amount of ground water ΔQ extracted from one side of the pocket is increased, the tongue depth y_c decreases. The limiting situation of $y_c = 0$ in each case produces a maximum water yield, which comes entirely from the pocket as the tongue no longer exists. For $y_c = 0$, $Q_t = 0$, hence $Q_\ell = \Delta Q$. By comparing values of Q_ℓ in Table 4 with corresponding values of ΔQ at $y_c = 0$ in Table 5, a substantial increase in flow through the pocket can be noted with the extraction. Qualitatively, this results from the reduction in piezometric level by the pumping, causing an increased downward gradient between the lens and the pocket.

CONCLUSION

The above analysis enabled the fresh-water flow in the lens and pocket as well as the interfaces to be determined for specified boundary conditions. Today, drainage ditches in the dunes create a much more complicated ground water table than described above with $z = f(x)$. Further calculations are necessary to take into account the discrepancies. Also the quantity N in actuality is more involved than described heretofore. It represents the difference between water supplied to the dunes by precipitation and artificial recharge

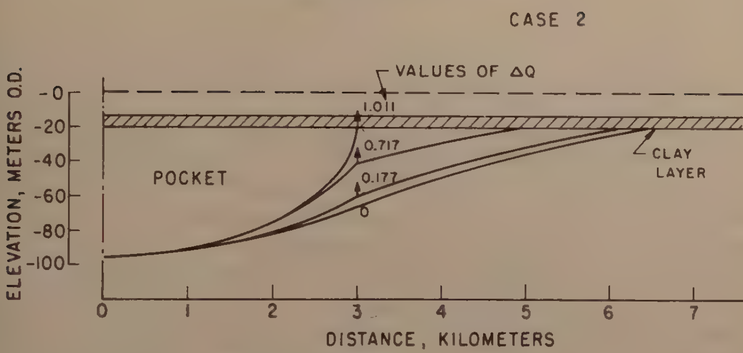

FIG. 10 FRESH-WATER DISTRIBUTION WITH PUMPAGE FROM THE POCKET, CASE 2

Rhine River water and water discharged from the dunes by evapotranspiration and drainage ditches. There is no surface runoff from the dunes.

Under present-day conditions the inland water table lies 5 meters below mean sea level. This induces a landward flow of salt water under the dunes which shifts the interface to destroy the symmetry of the pocket and tongues shown in Fig. 2. This factor increases the complexity of the problem to such an extent that no direct mathematical solutions can be obtained. The matter is being studied by engineers of the Municipal Water Works of Amsterdam; solutions by means of electronic computers appear to be possible.

Table 5 - Values of y_c and ΔQ

Case	y_c, m	ΔQ m^3/day	Case	y_c, m	ΔQ m^3/day
1	0	0.440	3	0	1.811
	10	0.327		20	1.517
	20	0.145		40	0.977
	26.30	0		60	0.220
				64.95	0
2	0	1.011			
	20	0.717	4	0	4.097
	40	0.177		20	3.803
	45.21	0		40	3.263
				60	2.505
				80	1.543
				100	0.382
				105.95	0

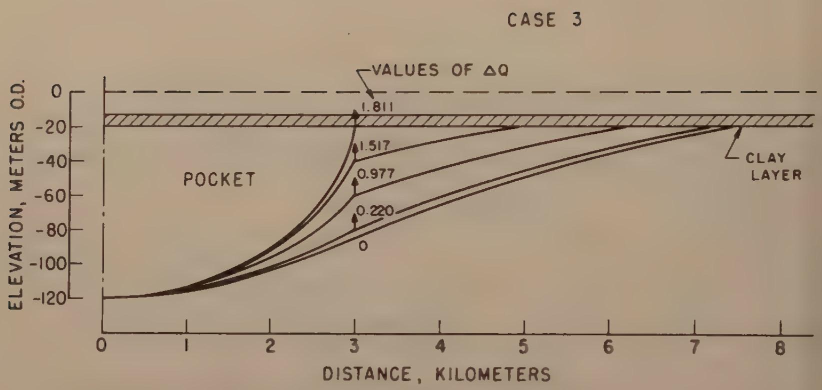

FIG. 11 FRESH-WATER DISTRIBUTION WITH PUMPAGE FROM THE POCKET, CASE 3

ACKNOWLEDGMENTS

Ir. C. Biemond, Director of the Municipal Water Works of Amsterdam, kindly assisted in the planning and general conduct of this study. Ir. K. D. Venhuizen, Engineer of the Municipal Water Works of Amsterdam, carried ou the calculations required for solving the various simultaneous equations. Participation in this study by the senior author was made possible by a post-doctoral fellowship from the National Science Foundation.

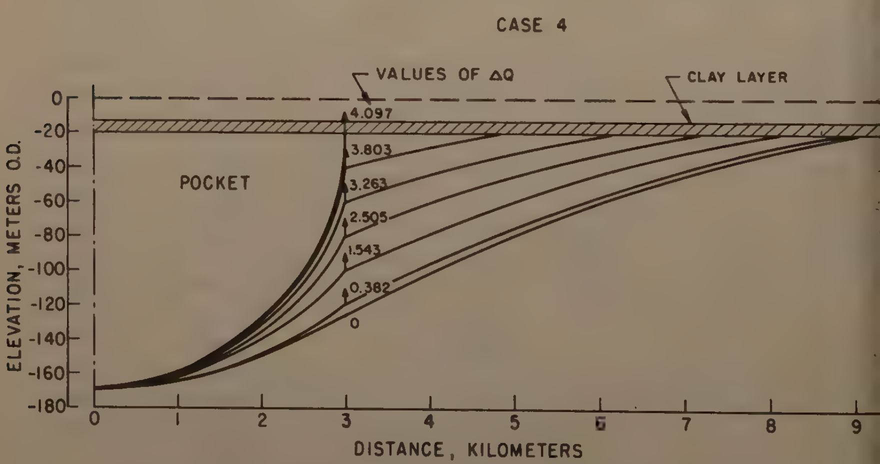

FIG. 12 FRESH-WATER DISTRIBUTION WITH PUMPAGE FROM THE POCKET, CASE 4

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A TIME INTERVAL DISTRIBUTION FOR EXCESSIVE RAINFALL

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ABSTRACT

Although recurrence intervals are of primary interest in frequency analysis of excessive precipitation, present methods average these intervals and thus conceal much design information. By relating probability to recurrence interval instead of rainfall depth the distribution of recurrence interval for preassigned rainfall amount is obtained. This provides additional information for design.

INTRODUCTION

The usual practice in obtaining design data for high intensity rainfall over small drainage areas is to express the frequency or mean recurrence interval as a function of depth.(1) The mean recurrence interval in years for a depth y is defined by the equation $R(y) = n/m$, where n is the length of record in years and m is the number of occurrences in the record equalling or exceeding the depth y. An occurrence equalling or exceeding y is called a y-event. E.g., in a 20 year record the second highest value has a mean recurrence interval of 20/2 or 10 years. The mean recurrence interval is also sometimes called the return period although the latter has usually been applied where only the extreme occurrence from each year is employed. Both frequency (or mean recurrence interval) and return period have the property that they give an average time interval of recurrence.

The averaging of the time interval between y-events has always seemed rather artificial, for the time interval appears to be the significant variable and averaging obscures much of the information it might provide. The effect of averaging has often been recognized by stating that the mean recurrence interval gives only the average recurrence period and hence y-events will

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often occur less than one mean recurrence interval apart in time. For example, a mean recurrence interval of 5 years for a y-event gives no measure of the risk that y-events will occur in two successive years. This seems to be a rather troublesome deficiency, for the purchaser of a drainage system would be little impressed by a mean recurrence interval if the probability of overloadings spaced one year or less apart were high and overloadings occurred in the first two years of operation. This deficiency will be overcome by defining a probability distribution on the time interval between y-events instead of averaging that time interval.

The Poisson Distribution

Although most are familiar with the ordinary Poisson distribution, the Poisson distribution with a time variable, which will be employed here, is not as well known and requires some further explanation.(2)

Consider a series of Bernouilli trials, viz. a set of independent trials with two possible events: a rainfall equalling or exceeding y, the y-event, with probability p, and a rainfall less than y with probability 1-p. The trials are assumed to be stationary i.e. p is constant for all trials. The probability of x y-events may be expressed by a Bernouilli or binomial distribution $B(x;n,p)$ in the usual manner where x is the number of y-events, n is the total number of trials, and p is the probability of a y-event. The y-events are considered to occur in a time sequence. Divide the unit time interval into n parts of length 1/n each. If n is large, either an interval contains a y-event or it does not; hence if successive unit intervals are non-overlapping and independent, the number of y-events x found in the unit intervals will be distributed in a Bernouilli distribution $B(x;n,p)$. Replacing the unit time interval by an interval of length t with the same subdivisions of length 1/n and n large again gives a Bernouilli distribution with the number of y-events and non-y-events in each t equal to nt to the nearest integer. The distribution then becomes $B(x;nt,p)$.

It is well known that the mean of the Binomial distribution is $\alpha = np$; so the mean of the time interval distribution is

$$(\alpha t) = ntp. \tag{1}$$

For small p, of interest here, the Bernouilli distribution approaches the Poisson distribution. In the case of the time distribution this may be seen as follows: Since $p = (\alpha t)/nt$, for small p, $1 - p \sim 1$ and

$$B(0;nt,p) = \left[1 - \frac{(\alpha t)}{nt}\right]^{nt}$$

Taking logarithms of both sides of this equation and expanding in Taylor's series gives

$$\ln B(0;nt,p) = nt \ln\left[1 - \frac{(\alpha t)}{nt}\right]$$

$$= -\left[(\alpha t) + \frac{(\alpha t)^2}{2nt} + \ldots\right].$$

For large n the series approaches $-\alpha t$, and

$$B(0;nt,p) \sim e^{-\alpha t}. \tag{2}$$

The general term of the Bernouilli distribution is

$$B(x;nt,p) = \binom{nt}{x} p^{nt}(1 - p)^{nt-x}.$$

For large n

$$\binom{nt}{x} \sim \frac{(nt)^x}{x!}.$$

Hence,

$$B(x;nt,p) = B(0;nt,p) \frac{(ntp)^x}{x!(1-p)^x}.$$

Substituting from Eqs. (1) and (2) and observing that $(1-p)^x$ approaches one gives

$$P(x; \alpha t) \sim e^{-\alpha t}(\alpha t)^x/x!. \qquad (3)$$

This is the Poisson probability of finding x y-events in a time interval t.

The Poisson distribution of Eq. (3) is best fitted to observational data by estimating α for a unit time interval. The nature of excessive rainfall and its use in design data make it appropriate to choose t equal to one year. With this assumption the parameter α has an optimal estimate a which is the arithmetic average of the number of y-events per year.

The a-values for one hour duration and various y-events for Omaha, Davenport, and Minneapolis are shown plotted in the figure and tabulated in Table 4. These a-values were employed in fitting the Poisson distributions tabulated in Tables 1, 2, and 3. This was done by simply substituting a for α and one for t in Eq. (3). In these tables x is the number of occurrences of a y-event in the year, g_o is the observed frequency, and P_o is the observed probability. P_c is the probability estimated from Eq. (3), g_c is the estimated frequency of occurrence, and F_c is the summation of P_c or the distribution function. To examine the goodness of fit of the Poisson distributions, g_c is compared with g_o. Simple inspection shows that there is reasonable agreement, and this is verified by X^2 values shown at the bottom of each table. All of these are exceeded in random sampling with a high probability indicating non-significant (N.S.) departure of the actual from theoretical frequencies and therefore a good fit by P_c. Hence the Poisson distribution provides a good fit to y-events of excessive rainfall. This is not surprising, for the excessive rainfall series quite naturally meet the basic assumptions for the Poisson distribution. For direct use P_c provides estimates of the probabilities for various x-values and F_c provides estimates of the probability that x is equal to or less than a given value. These results, however, are only supplementary to the main objective—the time interval distribution.

The Time Interval Distribution

The time interval t between y-events according to previous discussion is the random variable and has a distribution. This distribution can be found directly, but it suits the objective here best to develop it from the Poisson distribution.

Table 1
Poisson Distribution
Omaha, Neb., (1920 - 1955)

	g_o	P_o	P_c	g_c	F_c
0	8	.2222	.1643	5.9148	.1643
1	10	.2778	.2967	10.6812	.4610
2	6	.1667	.2680	9.6480	.7290
3	7	.1944	.1613	5.8068	.8903
4	3	.0833	.0728	2.6208	.9631
5	2	.0556	.0263	.9468	.9894
6	0	0	.0079	.2844	.9973
7	0	0	.0020	.0720	.9993
8	0	0	.0005	.0180	.9998

P ($^{2}_{3}$ 2.9725) 0.30; N.S.

Table 2
Poisson Distribution
Davenport, Iowa (1920 - 1955)

	g_o	P_o	P_c	g_c	F_c
0	4	.1111	.1787	6.4332	.1787
1	12	.3333	.3077	11.0772	.4864
2	12	.3333	.2650	9.5400	.7514
3	6	.1667	.1521	5.4756	.9035
4	2	.0556	.0654	2.3544	.9689
5	0	0	.0226	.8136	.9915
6	0	0	.0065	.2340	.9980
7	0	0	.0016	.0576	.9996
8	0	0	.0003	:0108	.9999

P ($^{2}_{2}$ 1.6327) 0.30; N.S.

Table 3
Poisson Distribution
Minneapolis, Minn. (1920 - 1955)

	g_o	P_o	P_c	g_c	F_c
0	15	.4167	.3679	13.2444	.3679
1	10	.2778	.3679	13.2444	.7358
2	7	.1944	.1839	6.6204	.9197
3	4	.1111	.0613	2.2068	.9810
4	0	0	.0153	.5508	.9963
5	0	0	.0031	.1116	.9994
6	0	0	.0005	.0180	.9999
7	0	0	.0000	.0000	.9999

P ($^{2}_{2}$ 2.4985) .20; N.S.

In the previous discussion it was shown that α y-events occur on the average per unit of time. If the time t is divided into a large number n of small intervals δt, it is seen that the probability of a y-event in each interval δt is $\alpha \delta t$. From Eq. (3) for the Poisson distribution it is seen that the probability of no y-event in the interval t is

$$P(0; \alpha t) = e^{-\alpha t}.$$

For the distribution of t the probability is required that there is no y-event in the n intervals of length δt comprising t and a y-event in the (n+1)th interval. This is a joint probability given by the product of probabilities of these events or

$$\delta p = e^{-\alpha t} \alpha \delta t.$$

Letting n increase without limit this equation approaches

$$dp = \alpha e^{-\alpha t} dt \tag{4}$$

which is the Poisson time interval distribution. The integral of this distribution over ranges of t gives probabilities of time intervals between y-events. Thus, the probability that the time interval shall be one year or less between y-events is given by the integral of Eq. (4)

$$F(1) = \alpha \int_0^1 e^{-\alpha t} dt \tag{5}$$

and in general for time interval T by

$$F(T) = \alpha \int_0^T e^{-\alpha t} dt. \tag{6}$$

It may be noted that only the integral values of T equal to or greater than one need to be considered in the excessive rainfall problem. Performing the integration in Eq. (6) gives

$$F(T) = 1 - e^{-\alpha T}. \tag{7}$$

By using $\underline{a}$ as the estimate for α, Eq. (7) provides all probabilities needed for recurrence intervals of length T. It should be noted that since the excessive rainfall series is independent and stationary, the good fit of the Poisson distribution to the data of Tables 1, 2, and 3 also insures a good fit to the time interval distribution and no direct test of fit of the latter is necessary.

It is of interest to see how the time interval distribution relates to the mean recurrence interval. Since Eq. (4) is the distribution of the recurrence interval, its mean value should be the mean recurrence interval. Writing the usual equation for the mean or expected value gives

$$\begin{aligned} E(t) &= \int_0^{\infty} t e^{-\alpha t} dt \\ &= \left[-\frac{e^{-\alpha t}}{\alpha^2} (\alpha t - 1) \right]_0^{\infty} \\ &= \frac{1}{\alpha}. \end{aligned} \tag{8}$$

In previous discussion it has been shown that the mean recurrence interval is the number of years divided by the number of y-events or n/m, whereas a is the number of y-events divided by the number of years of record or m/n. Hence, a and the mean recurrence interval are reciprocals of each other as verified by Eq. (8). From this equation it is also clear that much information has been ignored by only computing the mean recurrence interval 1/a. The time interval distribution may thus be viewed as a tool for analyzing excessiv rainfall data sufficient in itself, or as a means of extending an analysis which gives only the mean recurrence interval. For example, Yarnell(1) gives mea recurrence intervals called frequencies. They are estimates of $1/\alpha$ in the time interval distribution. Hence, Yarnell's results may be extended to recurrence interval probabilities by substituting the reciprocals of his frequencies for α in Eq. (7).

Another aspect of the relationship between the mean recurrence interval and the time interval distribution is apparent in Eq. (7). Rearranging (7), taking logarithms, and solving for α gives

$$\alpha = \frac{1}{T} \ln \left[\frac{1}{1 - F(T)} \right]. \qquad (9)$$

This relates the reciprocal of the mean recurrence interval to F(T) the probability of a recurrence interval of T or less and furnishes a means of appraising the mean recurrence interval in relation to a given F(T).

Applications to Observational Data

Considering its properties it appears that the mean recurrence interval is a fairly arbitrary choice of statistic for design purposes. An event which is expected to occur with an average time interval of a certain length throws little light on a problem in which the main concern is for the individual time intervals and particularly for the shorter intervals. As a consequence it appears that recurrence intervals other than the mean might be much more important than was apparent in the past. Perhaps much of the popularity of the mean recurrence interval is due to its long usage rather than its real merit.

From a practical design standpoint and also from the view of the purchase of certain types of designed systems, the shorter recurrence intervals might be the most important. Where risk of annual recurrence is particularly important the customer will be most anxious to be informed about this risk. In view of this it will be of interest to give certain statistics for the one and two year recurrence intervals.

For this purpose the a's of Table 4 were plotted in the figure and empirical straight lines drawn to the points on the assumption that the mean frequency decreases exponentially. The smoothed a-values of Table 5 were read from these lines for the various durations. The mean recurrence intervals which are the reciprocals of the a's are also shown there in parentheses. These agree well with Yarnell's results(1) estimated from quite a different period o record.

Using Eq. (7) with T = 1 Table 6 was computed giving the probabilities of the recurrence interval being one year or less. For the lower rainfall depths it is seen that the chances of annual recurrence is very high, and it is not until depths of 2.5 inches are reached that the probability of recurrence is reduced to a lower range of risk. Note from Tables 5 and 6 that a 0.05 risk of

Table 4
Sample a-values

One Hour Depth - inches	.75	1.00	1.50	2.00	2.50
Minneapolis	2.278	1.000	0.250	0.111	0.0278
Davenport	3.167	1.722	0.361	0.139	0.0556
Omaha	3.333	1.806	0.472	0.222	0.0556

Table 5
Estimated a-values and (Mean Recurrence Intervals)

One Hour Depth - Inches	1.00	1.50	2.00	2.50
Minneapolis	1.11(0.9)	0.323(3.1)	0.0938(10.7)	0.0275(36.4)
Davenport	1.70(0.6)	0.502(2.0)	0.146(6.8)	0.0427(23.4)
Omaha	1.85(0.5)	0.584(1.7)	0.186(5.4)	0.0584(17.1)

Table 6
Probability that T 1 year

One Hour Depth - inches	1.00	1.50	2.00	2.50
Minneapolis	.770	.276	.090	.027
Davenport	.817	.395	.136	.042
Omaha	.843	.442	.160	.057

Table 7
One Hour Rainfall Depth for F(T) = 0.05

	Depth for F(1)	Depth for F(2)
Minneapolis	2.25	2.53
Davenport	2.42	2.71
Omaha	2.56	2.86

annual recurrence is associated with about a 20 year mean recurrence interval. Using Eq. (7) this was found more exactly to be 0.0488. By use of this equation any given mean recurrence interval can be enhanced with the probabilities for any desired set of recurrence intervals with little effort.

The foremost objective of this analysis, however, was not merely to refin the mean recurrence interval or frequency analysis, but to present a method of analysis which might deserve consideration on its own merits. For this purpose the approach is the natural one of all design problems, i.e., to set th risk or probability in advance and then determine the numerical value of the design variable. This is done easily by means of Eq. (9) and the figure.

Much more practical experience with the application of the Poisson time interval distribution must become available before its usefulness can be appraised. However, as illustrations of possible application two examples for the lowest two recurrence intervals, one and two years, which would appear to be the most critical in design problems were chosen. In both examples 0.05 probability or risk of occurrence was assumed. Substituting 0.05 for F(T) and 1 and 2 respectively for T, α's of 0.0513 and 0.0256 were obtained from Eq. (9). These correspond to recurrence intervals of 19.5 and 39.0 years. Entering the lines of the figure with these α values the estimated depths of Table 7 were obtained. These are depths such that their one and two year recurrence intervals will only occur with probability 0.05. It is clear that the depths for two years must be less than those for one year sinc the former is a more stringent design condition.

From Table 6 it is clear that to insure with reasonable probability that th spacing of values exceeding design (y-events) shall not be less than one year design depths above 2.0 to 2.5 inches for one hour duration must be adopted. If the spacings between y-events are to be less than one and two years respectively with a probability of 0.05 or chance of one in 20, Table 7 gives the design depths which must be adopted. It would appear from these examples that in certain design problems where the most serious criticism arises fro the too close spacing of failures, such as exceeding the capacity of storm sewers, the only concern might best be for the probability of a specific recurrence interval rather than for a mean recurrence interval.

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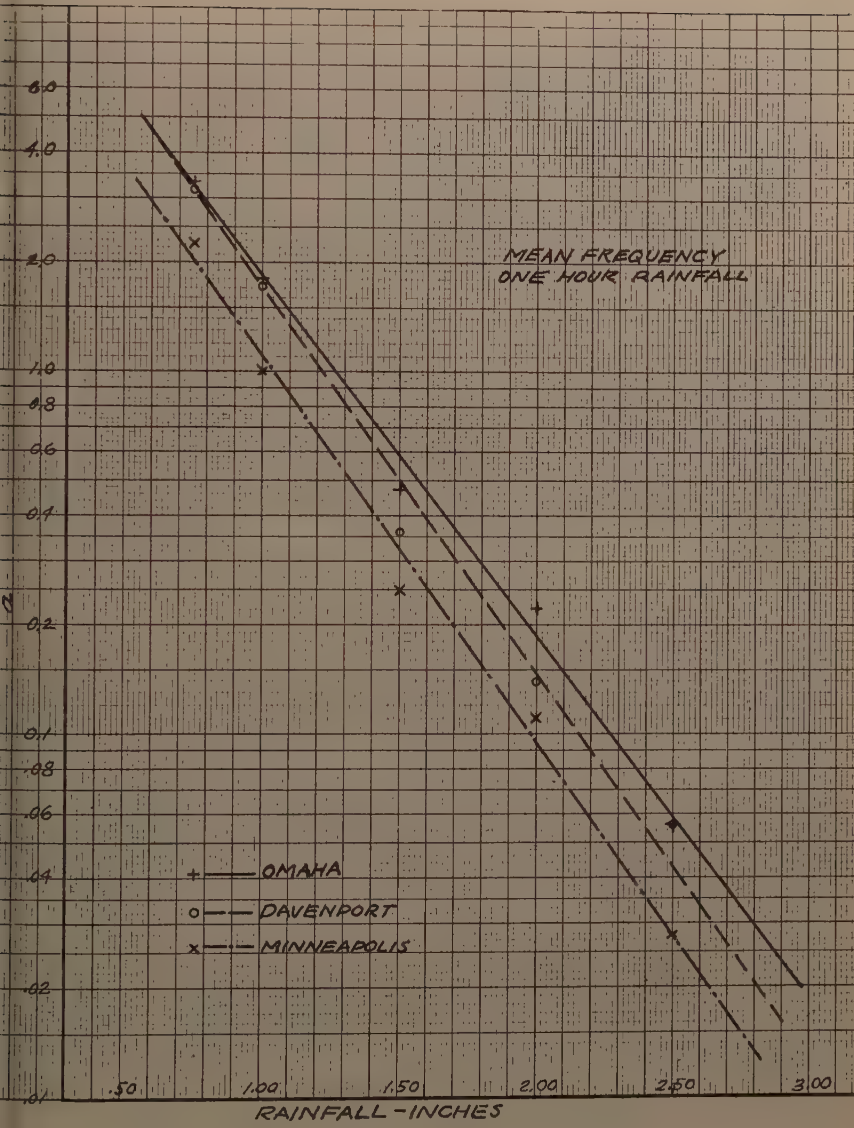
MEAN FREQUENCY
ONE HOUR RAINFALL
6.0
4.0
2.0
1.0
0.8
0.6
0.4
0.2
0.1
.08
.06
.04
.02
.01
OMAHA
DAVENPORT
MINNEAPOLIS
.50
1.00
1.50
2.00
2.50
3.00
RAINFALL - INCHES

Journal of the

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Proceedings of the American Society of Civil Engineers

DETERMINATION OF HYDROLOGIC FREQUENCY FACTOR

Ven Te Chow,[1] M. ASCE

SUMMARY

A graphical method for the determination of the frequency factor in a log-normal distribution of hydrologic data is described. It is found that the graphical procedure is much simpler and it produces results with less effort than the analytical method. The method described herewith is illustrated with a numerical example.

INTRODUCTION

In the design of a hydrologic engineering project it is almost always found necessary to know something of the probability of occurrence of hydrologic events of various magnitudes for which the structures of the project should be designed. From the viewpoint of safety, a structure in the project should be able to withstand the worst possible hydrologic event which might occur. From the point of economy, on the other hand, the cost of the structure must be justified by the returns to be expected from its construction. Consequently, the selection of a design hydrologic event involves a calculated risk (or probability) that a more severe magnitude of the event will not occur during the life (or design period) of the structure. In hydrologic design studies the knowledge of the probability of occurrence of hydrologic events is investigated by a frequency analysis through the use of statistical methods.

There are many methods of hydrologic frequency analysis in which the recorded magnitudes of a hydrologic event are taken as statistical variables and a statistical distribution of the variables is made to determine the statistical characteristics, including the frequency of a given magnitude of the hydrologic event. According to Chow,(1,2,3,4) most methods of hydrologic frequency analysis can be expressed by a simple general formula:

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1. Prof. of Hydr. Eng., Univ. of Illinois, Urbana, Ill.

$$x / \bar{x} = 1 + C_v K \tag{1}$$

in which x = the magnitude of a hydrologic event

$\bar{x}$ = the arithmetic mean of x

$x/\bar{x}$ = a dimensionless ratio of x to $\bar{x}$

C_v = the coefficient of variation of x

and K = the frequency factor of x for a given probability of occurrence.

This factor depends upon the pattern of the statistical distribution of x.

By elementary statistical procedures, the values of the arithmetic mean and the coefficient of variation can be computed from the historical data of th hydrologic event. The determination of the frequency factor, however, will depend further on the law of distribution of the hydrologic event and the assigned frequency (or probability) for the design. With all these values determined, the magnitude of the hydrologic event can be easily computed by (1) Conversely, if the magnitude of the hydrologic event is known, the frequency of the event can also be determined by (1). In this paper, the law of log-normal distribution is considered and the objective is to compute the corresponding frequency factor by a graphical method.

The Log-Normal Distribution and Related Formulas

When the logarithms of a variate x are normally distributed, the distribution of the magnitude x is said to be log-normal. The mathematical law which governs the log-normal distribution is called the log-probability law. It is generally found that hydrologic data of many kinds are log-normally distributed. A comprehensive discussion of the log-probability law and its engineering applications has been given by Chow.(5) The determination of the hydrologic frequency factor when a log-normal distribution is assumed has been presented,(6) in which the method employed is purely analytical. From these studies(5,6) a number of mathematical formulas have been developed, showing the relations between various characteristic values of the variate x that is log-normally distributed. The formulas which are found useful in the present graphical method are listed as follows:

$$C_s = 3\, C_v + C_v^3 \quad \ldots\ldots\ldots \tag{2}$$

$$C_v = \left[\exp(\sigma_y^2) - 1 \right]^{0.5} \tag{3}$$

or

$$\sigma_y = \left[\log_e (C_v^2 + 1) \right]^{0.5} \tag{4}$$

$$I_v = 0.4343\, \sigma_y \quad \ldots\ldots\ldots\ldots \tag{5}$$

$$\bar{x} / M = \exp(\sigma_y^2 /2) \quad \ldots\ldots\ldots\ldots \tag{6}$$

or

$$M = \bar{x} / \exp(\sigma_y^2/2) \quad \ldots\ldots\ldots \tag{7}$$

$$x' / M = \text{antilog}_{10}\, I_v \tag{8}$$

r

$$x' = M \exp \sigma_y \tag{9}$$

n these formulas the additional notation is as follows:

C_s = the coefficient of skew of the variate x,

y = $\log_e x$, the natural logarithm of x,

σ_y = the standard deviation of y,

I_v = the variability index, defined by (5),

M = the median of x which occurs at 50%-of-time,

nd x' = the magnitude of x exceeded 15.87% of the time.

The Method

The problem under consideration is to determine the frequency factors of he log-normally distributed variate x at various assigned probabilities when he value of either C_s or C_v is given. The graphical procedure for this de- ermination involves the following steps:

(1) When C_s is given, compute C_v by trial and error, using (2). It can be seen that it is much easier to compute C_s, using (2), when C_v is given.
(2) Compute σ_y using (4).
(3) Compute M by (7) assuming $\bar{x} = 1$.
(4) Compute x' by (9).
(5) It has been shown(5) that the probability at mean is that which occurs when the frequency factor of x is equal to zero and this probability is equal to

$$P_m = (1/\sqrt{2\pi}) \int_{K_y}^{\infty} \exp(-K_y^2/2)\, dK_y \tag{10}$$

where $K_y = \sigma_y/2$, the argument of the probability integral. Find the value of P_m which may be obtained from a normal probability function table for $K_y = \sigma_y/2$.
(6) On a log-normal probability paper such as shown in Fig. 1, represent the ordinate by $x/\bar{x}$, in which $\bar{x}$ is taken as unity, and plot any two of the following three points: (Ordinate = M, abscissa = 50%), (1, P_m), and (x', 15.87%.)
(7) Draw a straight line passing through the plotted points.
(8) On the plotted straight line read the value of $x/\bar{x}$ corresponding to the desired probability.
(9) For known values of C_v and $x/\bar{x}$, compute the corresponding frequency factor K using (1).

To illustrate the above procedure by a numerical example, assume $C_s = 1.139$. Then, by (2), $C_v = 0.364$. By (4), $\sigma_y = 0.3527$. By (7), M = 0.940. y (9), $x' = 1.338$. Since $K_y = 0.3527/2 = 0.1764$, the normal probability unction table gives $P_m = 43.0\%$.

Draw a straight line on the log-normal probability paper in Fig. 1, passing hrough the points: ($x/\bar{x} = 0.940$, P = 50%), (1.000, 43.0%), and (1.338, 5.87%). Actually, any two of the three points are sufficient for the con- truction of the straight line. From the straight line read the values of $x/\bar{x}$

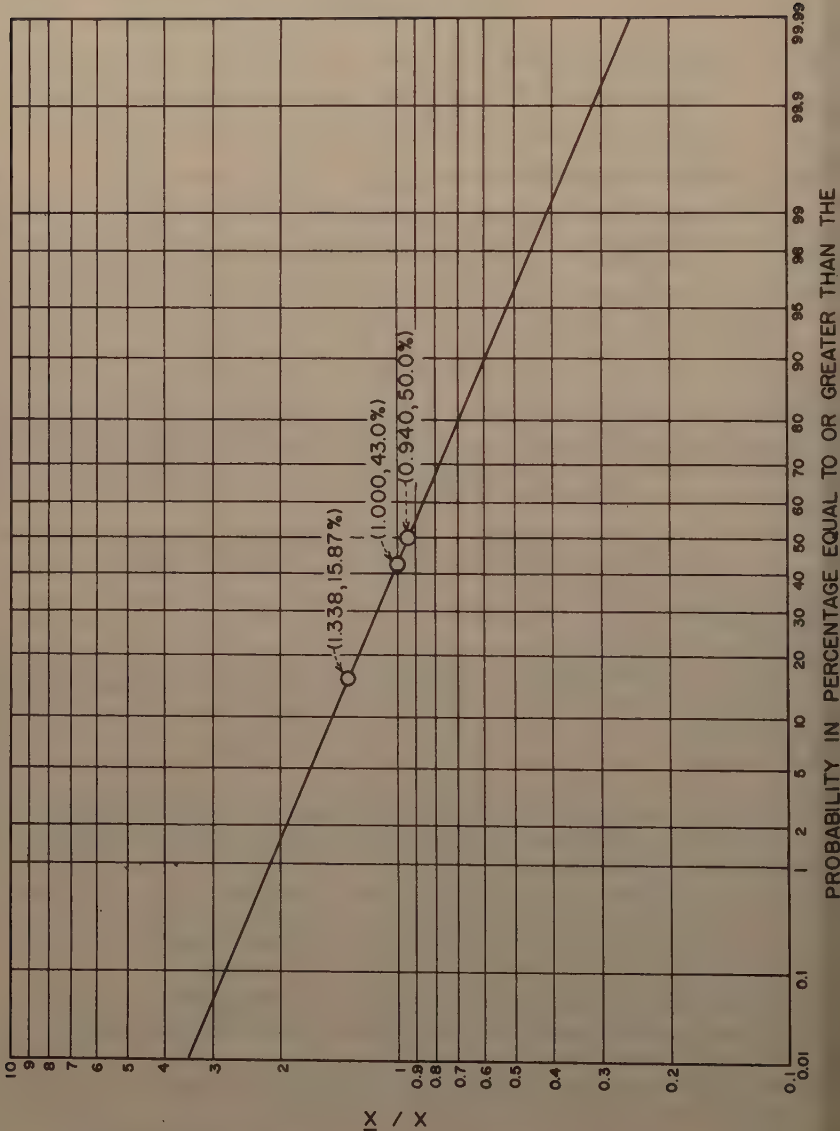

Fig.1 Graphical determination of log-normal frequency factors for C_s = 1.139

or various assigned probabilities equal to or greater than the value of $x/\bar{x}$. Using (1) with $C_v = 0.364$, the frequency factors are computed and listed in Table 1.

Table 1 — Computation of log-normal frequency factors for

$C_s = 1.139$ and $C_v = 0.364$.

	Probability in percentage Equal to or greater than the given variate									
	99.9	99	95	80	50	20	5	1	0.1	0.01
$(x/\bar{x})$ =	0.316	0.414	0.528	0.700	0.940	1.265	1.680	2.135	2.797	3.484
$(x/\bar{x})-1$ =	-0.684	-0.586	-0.472	-0.300	-0.060	0.265	0.680	1.135	1.797	2.484
K =	-1.879	-1.611	-1.297	-0.824	-0.165	0.728	1.869	3.119	4.937	6.824

A table of approximate log-normal frequency factors was first prepared by Hazen[7] and then revised.[8] Until recently, a corresponding table of theoretically exact log-normal frequency factors was computed by an analytical method.[5] It is believed that a great amount of time and effort can be saved for either preparing or expanding the table by using the graphical method described herewith.

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ROLL WAVES AND SLUG FLOWS IN INCLINED OPEN CHANNELS[a]

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SYNOPSIS

Roll waves and slug flows are established as two distinctly different wave atterns and are studied phenomenologically in an inclined open channel. The asic characteristics of flow are expressed in terms of pertinent physical roperties. A theory is presented and reference is made to similar phe- omena in allied fields in which mathematical analyses exist. It is hoped that is study will extend the present knowledge of unsteady phenomena in open annel flow.

INTRODUCTION

Flow in open channels is characterized by a free surface with constant ressure conditions. Possible steady state regimes have been designated onveniently as laminar-subcritical, laminar-supercritical, turbulent- ıbcritical and turbulent-supercritical. However, unsteady flows exist which re not amenable to analysis by methods applicable to any of the above re- .mes.

One such unsteady regime occurs in shallow flows in inclined open annels. It is characterized by intermittent surges and wavy patterns and as been variously called roll waves, rain waves, slug flows, and other names. ydraulic engineers encounter these unsteady flows in inclined open channels ıd spillways. The increased height of the waves requires additional free- oard to prevent spillage. The concentrated mass of these surges calls for lded structural safety factors against transient pressures and stresses. In

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laboratory studies of hydraulic models, these unsteady flows often interfere with similarity conditions.

To chemical engineers, the phenomena are important in liquid-gas reactic processes. Liquid mass transfer rate is basic to diffusion reactions and the rate is greatly increased in unsteady regions.

As a result of this investigation, two distinctly different wave train phenomena can be delineated. They are designated respectively as roll waves and slug flows.

Roll waves are the result of the interaction of surface tensions and gravit forces at a slightly disturbed surface. They are characterized by transverse ridges of high vorticity and intermittent quiescent zones (Plate I).

Slug flows result from instabilities which cause the transition from super critical laminar to turbulent flow. Locally disturbed regions spread transversely and contaminated adjacent zones similar to turbulent spots in wind tunnel investigations. Slug flows are characterized by a succession of highly agitated surges separated by turbulent regions (Plate II).

The Theory of Roll Waves and Slug Flows

Surface disturbances on a liquid body are due to externally impressed disturbances on amplified internal perturbation. A surface disturbance is subje to both surface and gravity forces.

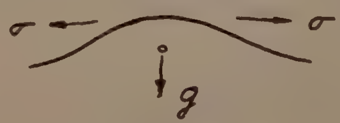


Fig. 1

When set in motion, capillary and gravity waves result. The general equation for the celerity of surface waves contains expressions for both types.(2)

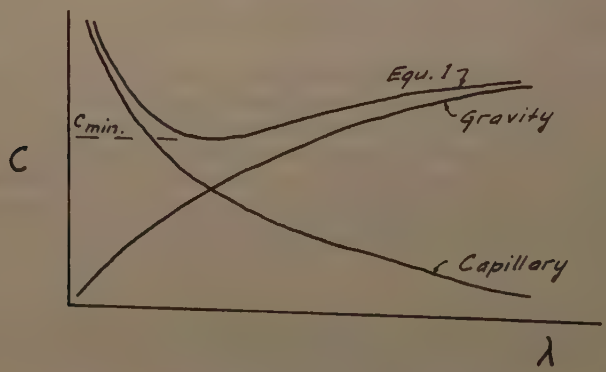


Fig. 2

Plate I. Roll Waves

Plate II. Slug Flows

$$C = \sqrt{\frac{2\pi\sigma}{\rho\lambda} + \frac{\lambda g}{2\pi}} \tag{1}$$

As indicated in Fig. 2, for waves of short wave length and large curvature, capillary waves predominate. Long waves are predominantly gravity waves and in shallow flows, their celerity approaches $\sqrt{gD}$. Celerity is defined as the wave speed relative to the medium. A minimum celerity exists for surface waves when both influences are equal. C_{min} is obtained by differentiation of the general wave equation in respect to λ. Then,

$$C_{min.} = \sqrt{\frac{4g\sigma}{\rho}} \tag{2}$$

and the corresponding wave length is

$$\lambda = 2\pi\sqrt{\frac{\sigma}{\rho g}} \tag{3}$$

When small disturbances are impressed upon a surface, the celerity of each wave is determined uniquely for every wave length. However, wave groups are known to travel at speeds different from the celerity of solitary waves. The group velocity of capillary waves is

$$C_{group} = 3/2\ C \tag{4}$$

and for gravity waves

$$C_{group} = 1/2\ C \tag{5}$$

Both expressions are derived from the general form

$$C_{group} = C - C\frac{dC}{d\lambda} \tag{6}$$

Roll waves and slug flows are complex phenomena and do not conform strictly with the group theory.

In open channel flow with a disturbed surface, both capillary waves and gravity waves exist. Their behavior and the competition between them are responsible for the formation of roll waves and slug flows. Most solutions of instability problems and the analysis of progressive waves are based upon the assumption of parallel flow. In this study, the proximity of the boundary is important and the initially smooth flow suggests viscous flow. For a case of one-dimensional viscous motion, the Navier-Stokes equation reduces to

$$0 = \nu\nabla_y^2 + g\sin\theta \tag{7}$$

Assuming a no-slip condition at the fixed boundary

$$v = 0 \qquad \text{at } y = 0$$

and

$$\mu\frac{dv}{dy} = 0 \qquad \text{at } y = D$$

Upon integration, the velocity becomes

$$v = \frac{g\sin\theta}{\nu}\left(yD - y^2/2\right) \tag{8}$$

At the surface, y = D, and

$$v = V_s = g\frac{D^2 \sin\theta}{2\nu} \tag{9}$$

This profile is obviously parabolic and the mean stream velocity is

$$V = \tfrac{2}{3} V_s = g\frac{D^2 \sin\theta}{3\nu} \tag{10}$$

For relatively shallow slopes

$$\sin\theta \cong \tan\theta \cong S$$

and, hence,

$$V = g\frac{D^2 S}{3\nu} \tag{10a}$$

For progressive waves to form, the following condition must be satisfied

$$V + \sqrt{gD} \gtrless V_s \tag{11}$$

For laminar flow, a limiting condition exists when an equality is assumed

$$V + \sqrt{gD} = V_s = \tfrac{3}{2} V \tag{12}$$

Subtracting from both sides the mean stream velocity and dividing by $\sqrt{gD}$, the result is

$$1 = \frac{1}{2} N_F$$

and

$$N_F = 2 \tag{12a}$$

Interestingly, this critical value of Froude number was also established by M. T. Lighthill and G. B. Whitham,(3) who used the Chezy equation in their derivation.

Other significant parameters are the slope and Reynolds number. Using expression (10a), the Reynolds number becomes

$$R_D = g\frac{D^3 S}{3\nu^2} \tag{13}$$

The velocity and depth expressed in terms of Reynolds numbers become

$$V = \left(\frac{gS\nu}{3}\right)^{1/3} R_D^{\,2/3} \tag{14}$$

and

$$D = \left(\frac{3\nu^2}{gS}\right)^{1/3} R_D^{\,1/3} \tag{15}$$

The critical Froude number can now be interpreted in terms of a Reynolds number and a slope. From expressions (12a), (14), and (15)

$$\frac{1}{2}\left(\frac{gS\nu}{3}\right)^{1/3} R_D^{\,2/3} = \sqrt{g\left(\frac{3\nu^2}{gS}\right)^{1/3} R_D^{\,1/3}} \tag{16}$$

Squaring both sides and simplifying, the relationship becomes

$$R_D = \frac{12}{S} \tag{17}$$

and

$$S = \frac{12}{R_D} \tag{17a}$$

Similarly, in laminar flow, the Froude number is

$$N_F = \frac{g D^2 S}{3 \nu \sqrt{gD}} \tag{18}$$

and in terms of Reynolds number and slope

$$N_F = \sqrt{\frac{S}{3}} \sqrt{R_D} \tag{19}$$

which again leads at $N_F = 2$ to

$$S = \frac{12}{R_D}$$

As a matter of interest, the critical slope formula by Thomas,(4) Dressler,(5) Lighthill(6) and others was

$$S = \frac{4g}{C_1^2}$$

where C_1 was the coefficient of Chezy's formula. Now, the Chezy formula can be expressed for laminar flow as follows

$$V = g\frac{D^2 S}{3 \nu} = C_1 \sqrt{gD} \tag{20}$$

$$V = g\frac{D^{3/2} S^{1/2}}{3 \nu} \sqrt{DS} \tag{20a}$$

$$V = \sqrt{\frac{g}{3} R_D} \sqrt{DS} \tag{20b}$$

Hence,

$$C = \sqrt{g/3\, R_D} \tag{21}$$

Inserting this coefficient into the critical slope formula gives

$$S = \frac{4g}{C^2} = \frac{4g}{g/3\, R_D}$$

and significantly,

$$S = \frac{12}{R_D}$$

The behavior of surface disturbances on flows with Froude numbers is

$$N_F \geqq 2$$

next considered. The criterion was established by assuming the surface flui velocity equal to the velocity of a small wave so that the profile remains unaltered and the waves cannot break.

If the Froude number is less than $N_F = 2$, the wave velocity exceeds the surface velocity of the stream and

$$V + \sqrt{gD} > V_s$$

The initial disturbances may be sinusoidal pulses, so that both humps and depressions exist. The propagation speed of long waves increases with increases in height. Also, the higher points of a wave profile move with great speed than lower points.(7) Thus, waves steepen until breaking is imminent (Fig. 3).

Fig. 3

The increase in curvature brings more surface tension effects into action Fig. 4 shows how long gravity waves decelerate as capillary forces become important. In the limit, the condition of minimum celerity is attained.

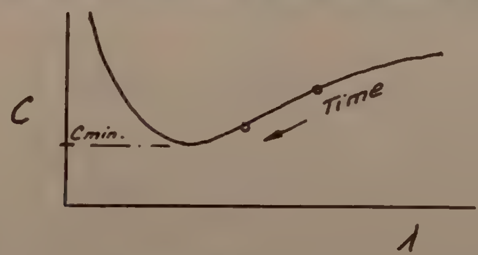


Fig. 4

As the wave length decreases the wave height increases and breaking of progressive waves in shallow water always takes place.

The frontward steepening of gravity waves and the subsequent increase of capillary effects are primarily responsible for roll wave formation. Two possible developments are suggested:

Case 1

Long waves steepen, decelerate, amplify and break. If the resulting wave are influenced equally strong by capillary and gravity forces, a wave train of ripples results which moves continually with a constant velocity.

$$V_w = V + C_{min.}$$

Case 2

When the breaking of the waves takes place, a spectrum of wavelets results. Each in turn is subject to surface and gravity forces. The wavelets a

sumed to be of the same height but not necessarily of the same frequency. nce the wavelets may have celerities different from the initial wave, some e assumed to interfere with each other. This interference is similar to the enomenon of sound waves which have different frequency and phase re- tions. The resultant of two waves of nearly the same frequency varies in nplitude from zero to twice the amplitude of either component. In the annel, the process of interference may repeat itself until waves of larger nplitude and selected frequency predominate. Finally, the forces of gravity ceed the surface tension effects. The larger waves overtake smaller waves, alesce with them and grow even larger. After a formative period, roll ves travel independently of each other and approach a terminal velocity ig. 5).

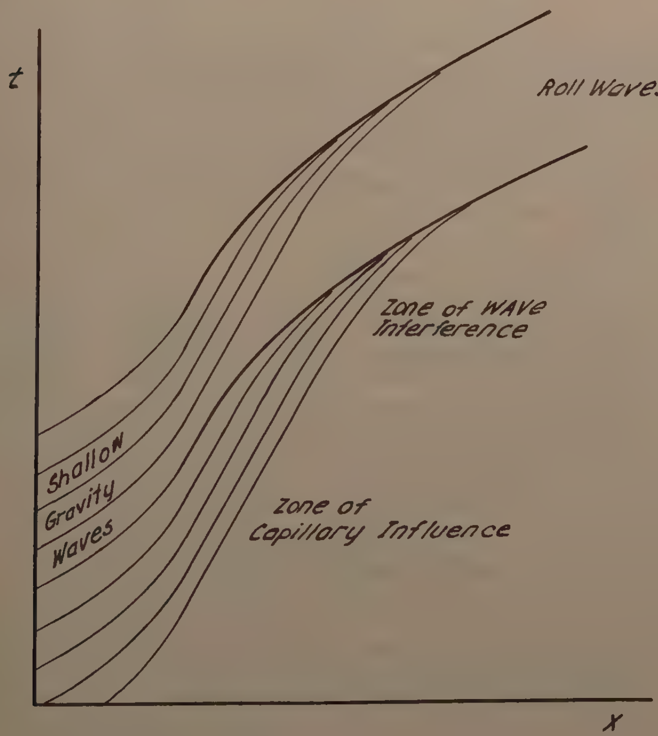


Fig. 5. Development of Roll Waves

When the Froude number is larger than two ($N_F > 2$), the velocity of a nall surface wave is less than the surface velocity of the fluid. The steepen- g of the wave occurs at the upstream end. The increase in curvature causes decrease of wave length and a deceleration of the wave, and consequently, e waves break at the upstream end. (Fig. 6)

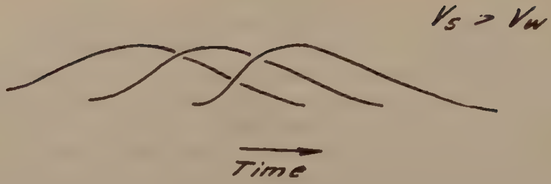


Fig. 6

The subsequent developments are outlined by two hypothesis.

Hypothesis 1

The waves break at the upstream end and the entire stream is affected. Since the wave velocity is less than some fluid velocity, the higher velocity fluid enters a region decelerated by the breaking of the wave. Consequently, the main stream is also decelerated. Some of the kinetic energy is converte into potential energy and the depth of flow increases. Some of the kinetic energy is expended in agitation and mixing. The combined effects are a loca hydraulic jump which marks the transition from smooth supercritical flow t highly agitated flow.

Hypothesis 2

The breaking of the wave unbalances the dynamic equilibrium of the supe critical stream which is potentially ready to become turbulent. In laminar supercritical flow on a smooth channel, the breaking of waves triggers the transition to turbulent flow. The change in flow conditions exhibits a sudden increase in resistance to flow. Although the channel slope and the boundary conditions are unaltered, the high velocity flow enters upon a friction slope not sufficient to maintain that velocity and a hydraulic jump occurs.

Subsequently, the agitated region grows into adjacent undisturbed flow. The velocity distribution in each regime is different and mixing takes place. Vorticity transport and turbulent agitation cause lateral contamination. The jump front is maintained as long as it is fed energy by the entering high velocity stream (Fig. 7). When an earlier jump intercepts this high velocity stream, the jump front remains a negative bore until all available energy is fed into it. Then the jump decays and forms an expansion wave and the dow stream face of the disturbed region becomes a positive bore. Subsequent bores telescope and form the characteristic saw-tooth profile of slug flows (Figs. 8 and 9).

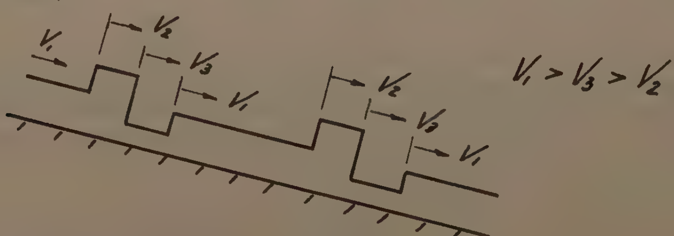


Fig. 7

Fig. 8

Laboratory Experiments

Rigorous mathematical treatment of complex hydraulic phenomena encounters great difficulties. Convenient approximations often lead to equations of questionable veracity. The background of experimental reality provides important insight into physical phenomena. Interpretive physical reasoning and dimensional analysis are other tools important to an investigator.

The pertinent variables of a hydraulic phenomenon may be categorized into:

a) Boundary conditions,
b) Kinematic and dynamic flow characteristics,
c) Fluid properties.

A general equation of flow may be written such that a function of all the variables is set equal to zero. Then, suitable power products of the variables can be grouped into dimensionless parameters.[8] Quantities pertinent to the initiation and subsequent development of roll waves and slug flows were:

D_o,	depth upstream from disturbance
D,	depth at point of initiation of disturbance
D_h,	depth of average wave crest
X,	channel station at point of initiation
S,	slope of channel
q,	volume rate of flow per unit width
Vs,	surface velocity of water
V_w,	velocity of roll wave
V_j,	velocity of traveling hydraulic jump (turbulent spot)
V_{sl},	velocity of slug flow
ρ,	mass density of water
σ,	surface tension
ν,	kinematic viscosity
f,	frequency of waves
λ,	wave length

All quantities are expressed in basic dimensions of the English system. The flow characteristics and boundary conditions were measured in the laboratory. Fluid properties were taken from appropriate tables.

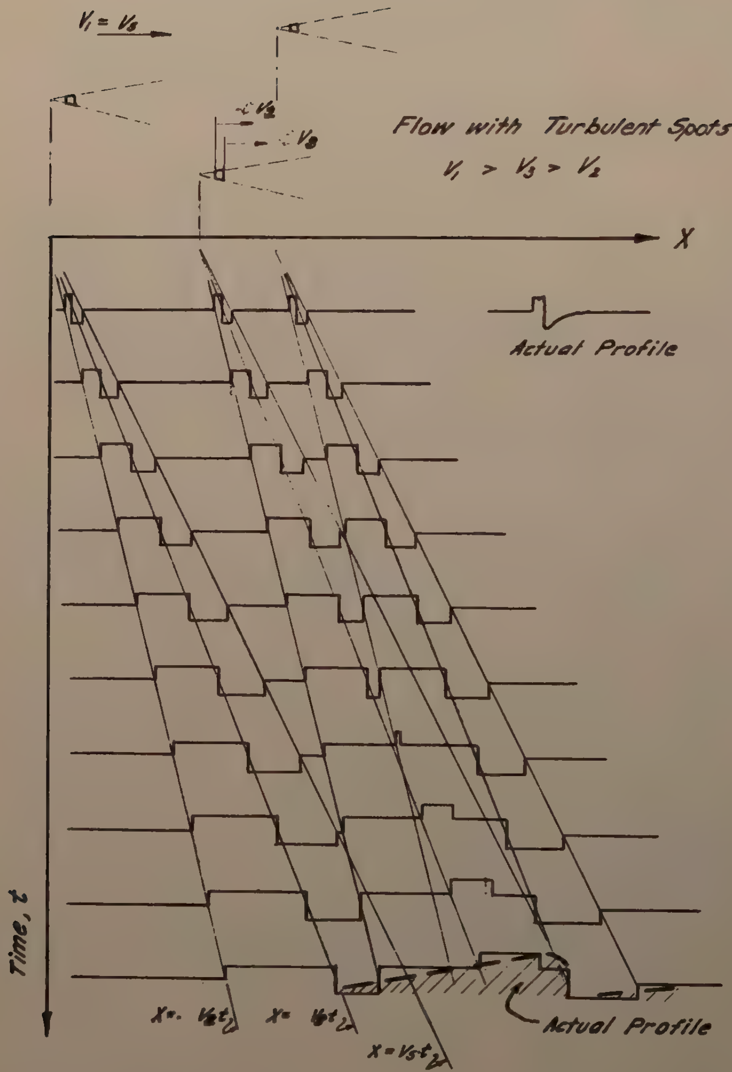


Fig. 9. t-X Diagram with Model of Slug Flow Development

Laboratory Equipment

Apparatus used in this experiment included the test flume; equipment to measure discharge, depth, and velocity; a disturbance generator; and photographic equipment. Some of the apparatus was assembled by the author with this specific investigation in mind. Other equipment was existent and modified.

The experimental work was carried out with a tiltable flume (Fig. 10 and Plate III). The channel consisted of two sections. Each section was 12 feet long, 18 inches wide, and 12 inches deep. The channel floor was covered by two 12 foot long slabs of 1/4 inch thick plate glass. The side walls, also of plate glass, were installed in 4 foot sections. The underside of the floor slabs were painted black. The side walls were left clear. All longitudinal and transverse joints were cast with paraffin and planed down to perfect joint conditions.

The tilting of the channel was accomplished by adjusting the various jacks manually.

Water entered the test flume through a 3-1/2 by 2 by 1-1/2 foot head tank. Wire screens were placed in the tank which supported a graded gravel and sand layer four inches thick. This filter course was intended to reduce the initial turbulence in the flow. A curved steel section extended into the stilling basin. It was aligned with the glass bottom in order to obtain smooth entrance conditions. The top members of the channel frame served as supports for continuous brass rails of 3/4 inch diameter. This track facilitated the quick and easy movement of an instrument carriage. The channel was marked in one foot stations, commencing at the crest.

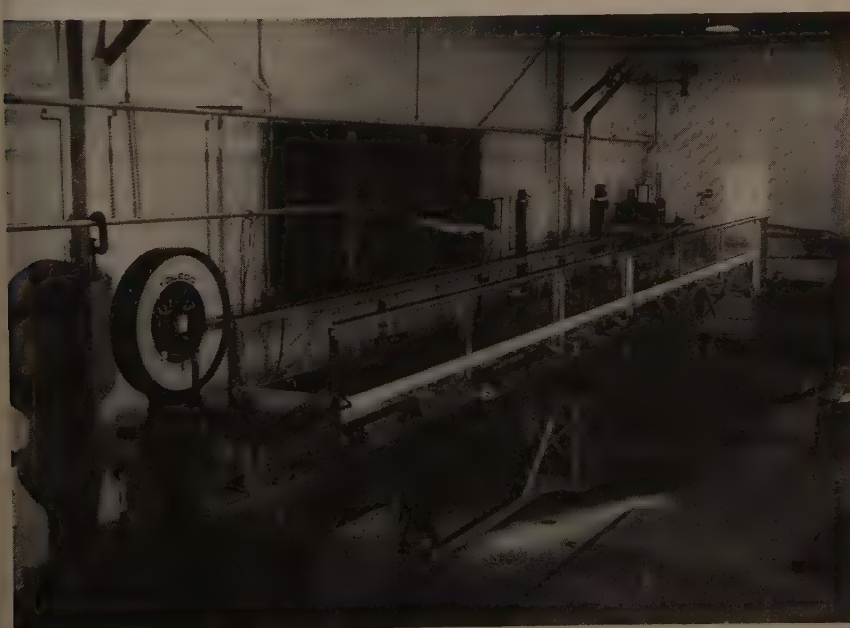


Plate III. Experimental Channel

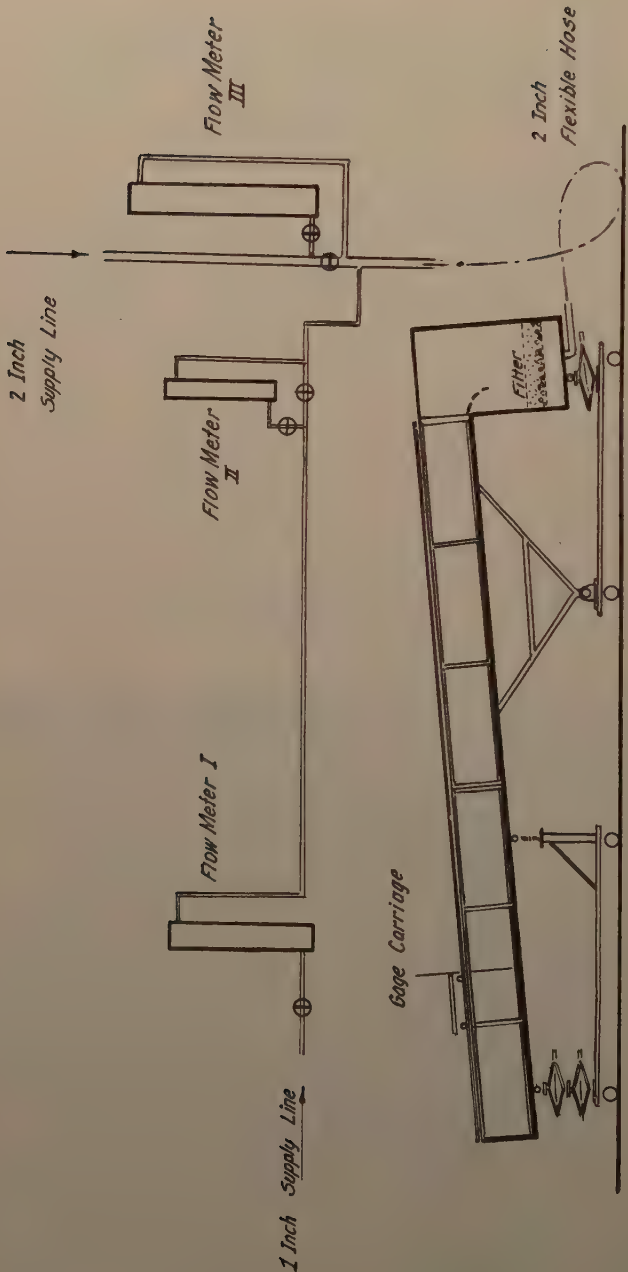


Fig. 10. Sketch of Laboratory Installation

The water supply was measured by a series of three float type variable area flow meters. Their respective effective ranges were:

a) From 0.001 cfs/ft. to 0.006 cfs/ft.
b) From 0.005 cfs/ft. to 0.030 cfs/ft.
c) From 0.010 cfs/ft. to 0.060 cfs/ft.

These meters had been calibrated and were checked intermittently by collecting the flow for gravimetric measurements. A Toledo dial type platform scale was used. All water was subsequently wasted.

Depth measurements under steady flow conditions were made with a point gage. This gage was mounted on the instrument carriage. For greater accuracy, an attached Federal dial indicator allowed readings to 0.001 inches.

For depth measurements in wavy and unsteady flows, the variations in height were recorded by the electronic wave recorder. The primary component of this consisted of two 0.005 inch thick platinum probes separated by a distance of about sixteen millimeters (5/8 inch). The passing water acted as the electrolyte. The signal was amplified in the wave height measuring device. This amplifier had been built by the Instrumentation Branch of the U. S. Army Corps of Engineers, Waterways Experimentation Station, Vicksburg, Mississippi. A direct writing Brush oscillograph recorded the variations in depth (Fig. 11). The oscillograph records were readily converted into depth measurements with the aid of a calibration curve. The accuracy and reproducability of the electronic measurements were most satisfactory. The Brush oscillograph could be operated at three different speeds: 5 mm./sec., 25 mm./sec., and 12.5 cm./sec. The slowest speed was generally used to measure wave heights and frequencies. The faster speeds were more able to show wave profiles.

For the determination of surface velocities and wave velocities, the time measurements were accomplished with an electric clock capable of recording 0.01 sec. The clock was activated manually by means of a microswitch. The switch was attached to a long extension cord and thus permitted utmost mobility. The channel stationing was used for length measurements.

A small bottle of compressed gas was attached to the head wall of the stilling basin. A flexible hose connected the bottle to a control valve located in the head wall below the water surface. The rate of release of gas bubbles could also be regulated by the valve attached to the gas container.

All pictures were taken by the author with a Kodak Graphic View camera. Royal Pan 4 by 5 inch sheet film was used exclusively.

Laboratory Procedures

The laboratory procedures evolved during a period of preliminary work. Necessary precautions were detected early and followed in subsequent experiments. Precursory investigations delineated the ranges of slope and discharge within which satisfactory measurements could be taken. All pertinent channel and flow characteristics were observed and recorded repeatedly.

Five different channel slopes were used. Their respective tangents were 0.0175, 0.0349, 0.0524, 0.0699, and 0.0875. On each slope, a series of nine discharges was investigated which ranged from 0.001 cfs/ft. to 0.06 cfs/ft. After the channel slope was set and a given discharge obtained, the instrument carriage was moved from station to station. Considerable time was allowed to elapse after each movement of the carriage in order to eliminate external

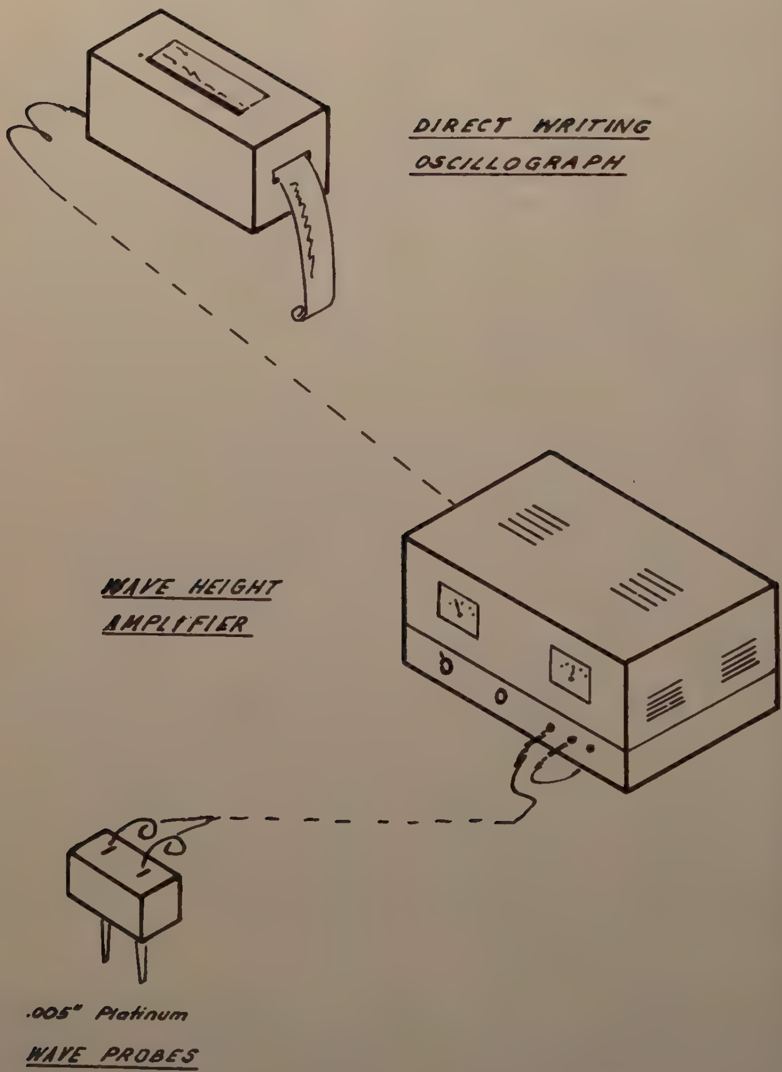


Fig. 11. Schematic View of Wave Recorder

disturbances as far as possible. Depth measurements and wave recordings were then made. Test results obtained under these conditions were recorded as Regime "Q" (quiescent).

The mean velocity of the flowing water was calculated from the known conditions of discharge and depth. Surface velocities were measured by allowing specks of talcum powder to travel between given stations and by measuring elapsed time. This process was repeated four or more times for each measurement.

Wave velocities were established by measuring the time interval in which a wave crest moved between predetermined stations.

The entire sequence of measurements was repeated with the disturbance generator in operation. Test results under these conditions were reported as Regime "S" (shock).

The initial location, type, and subsequent development of the surface disturbances were observed and recorded for every setting of slope and discharge as well as Regime "Q" and Regime "S". Photographs of significant events were taken. Best photographic effects were obtained by indirect lighting. Thus, waves appear in most photographs as shadows.

The water temperatures were measured both in the entrance tank and at the discharge end.

Results of Experiments

The presentation and analysis of the experimental results is best accomplished by a subdivision of the material into a logical sequence. First, the flow conditions leading to instability are discussed. Then, roll wave and slug flow characteristics are presented.

Discussion of Instability

Unstable Flows Leading to Roll Wave Formation

In the experimental flume, perfectly smooth laminar sheet flow degenerated into a flow marked by transverse ridges. The instability occurred in a flow which was initially undisturbed. It can be attributed to a natural mode of oscillations inherent in the flow. The same physical results were obtained by the superposition of finite disturbances by means of the disturbance generator. The distinctive difference between the operating conditions was that the breakdown of the flow took place further upstream when finite disturbances were externally impressed.

External disturbances of various amplitudes and frequencies were used. Both factors seemed to bear upon the stability phenomenon. However, the breakdown occurred sooner with larger amplitudes. No quantitative measurements of the disturbance amplitudes or frequencies were made. Significantly, the instability of flow, and hence the formation of roll waves, was eliminated when a wetting agent was added to the water. A wetting agent may affect not only the surface tension of the liquid but also cohesion and spreading. Since spreading of a liquid on a solid surface takes place when the work of adhesion exceeds that of cohesion, an inter-relationship exists between surface tension, cohesion, and spreading.(9) The exact action of a specific wetting agent upon the behavior of a liquid in motion requires considerably more study.

Plate IV. Vorticity in Crest of Roll Waves

The reduction of surface tension eliminated an essential agent for the for-
nation of roll waves. The fact that instability of flow and formation of roll
vaves were not observed when a wetting agent was added to the fluid is helpful
o hydraulicians engaged in model work.

After the breakdown of the laminar flow was precipitated, the development
of roll waves proceeded identically, regardless of the mechanism which
caused the instability of flow. Hence, laminar flow in inclined open channels
was unstable because of inherent disturbances as well as externally impressed
perturbations. Surface tension played an important role.

Jnstable Flow Leading to Slug Flow Formation

Instability leading to the formation of slug flows is more properly de-
scribed as the transition from laminar to turbulent flow. Instantaneous point
disturbances occurred randomly in the smooth sheet. The disturbed regions
spread transversely and were swept downstream at the same time. The
photograph on Plate VI shows this phenomenon. The highly agitated regions
differentiated the turbulent flow from the surrounding stream. The nature of
these agitated zones was similar to oblique traveling hydraulic jumps. In the
iterature of aerodynamics, these transitions are known as turbulent spots.[10]

Superimposed disturbances from the disturbance generator had no appreci-
able effects upon the stability of the flow. However, raindrops onto the
channel and sand grains in the channel precipitated turbulent spots. The ad-
dition of a wetting agent to the water had no detectable effect upon the flow
stability.

Analysis of Experimental Data of Initiation Conditions

Pertinent boundary and flow characteristics of the undisturbed flow prior
o the breakdown were grouped into the following five dimensionless parame-
ers:

Depth Reynolds Number,	q/ν	, R_D	(22)
Length Reynolds Number,	VXi/ν	, R_X	(23)
Froude Number,	$V/\sqrt{gD}$	, N_F	(24)
Weber Number,	$\rho V^2 D/\sigma$	, N_W	(25)
Slope,	$\sin\theta \sim \tan\theta$, S		(26)

In the presentation of the various characteristic curves, specific symbols
were selected to represent certain flow conditions. A legend of the symbols
and their meaning is given below:

○ , ripple flow, a train of small waves with short wave length,

△ , roll waves,

✳ , slug flows,

⌖ , confused pattern,

+ , agitated turbulent flow, usually at high Reynolds numbers. $R_D > 4000$,

△̶ , roll waves at verge of breakdown, usually at $R_D \sim 420$.

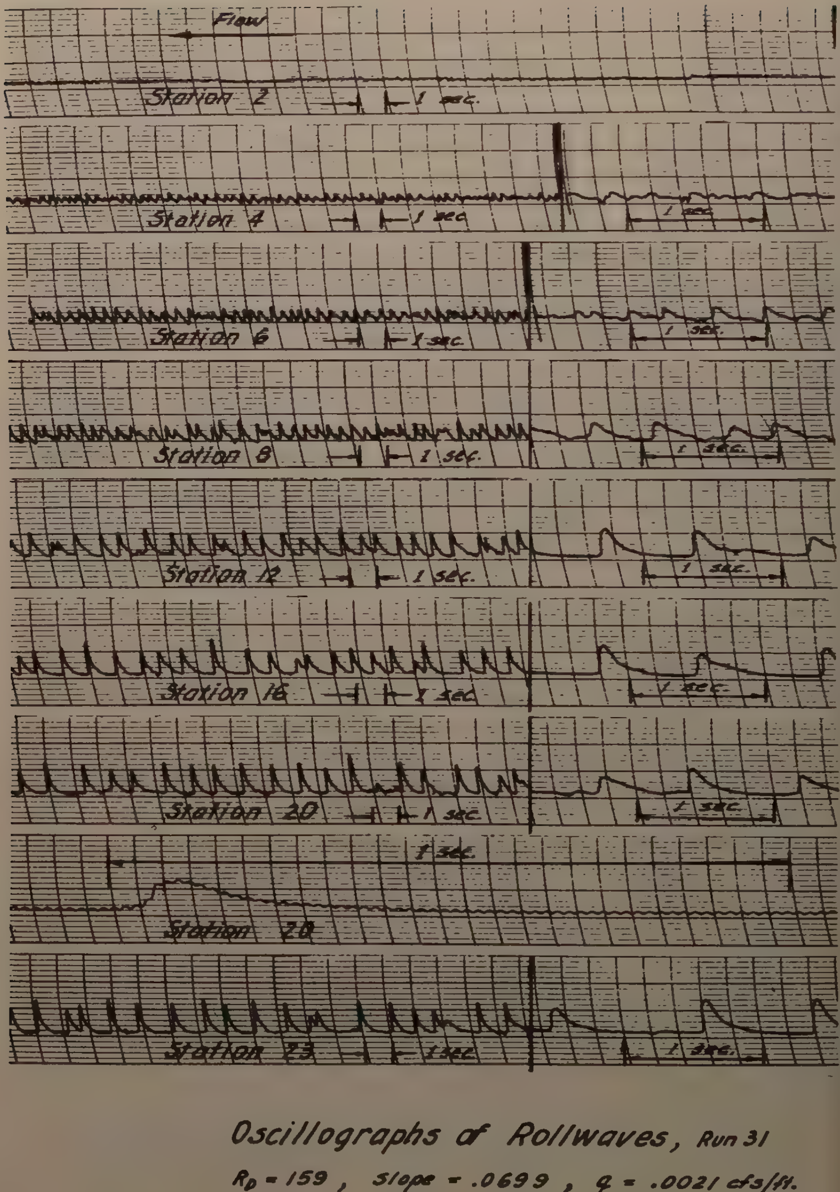


Plate V. History of Roll Waves

Plate VI. Turbulent Spots

A plot of Froude numbers and Reynolds numbers is presented in Fig. 12 with the slope as the third parameter. An analogous relationship was obtained when the gravitational and the surface forces were compared (Fig. 13). The transitional character of these curves is obvious. The graphs on Figs. 12 and 13 show that the surface tension and viscous effects predominated in the phenomenon of roll wave formation. At higher Reynolds numbers, corresponding to the conditions which led to slug flows, viscous and surface tension effects decreased and, finally, became negligible.

From the theoretical considerations, it was seen that the mean velocity and the depth of flow were a function of the slope. Thus, a simplified parameter, the modified Froude number, $N_F/\sqrt{S}$, and the Reynolds number could be plotted into a single curve (Fig. 14). Similar conditions led to a single representation of Froude number, Weber number, and slope (Fig. 15).

A combination of all parameters was undertaken so that an overall grasp of the instability phenomenon might be obtained. This also has practical engineering importance. In practical problems, one may know the conditions of slope, discharge, and temperature. Hence, a parameter exclusive of depth was plotted against a parameter which was a function of depth, slope, and temperature. The dimensionless parameter containing the rate of flow, q, was named "critical flow number", F_{cr}, the one containing the depth of flow, D, was named "critical depth number", N_D. The derivation of these parameters is shown below.

The instability condition was considered a function of R_D, N_F, N_W, and slope. Thus,

$$0 = \phi\left[R_D, N_W, N_F, S\right] \tag{27}$$

which is identical to:

$$0 = \phi\left[\frac{q}{\nu}, \frac{\rho q^2}{\sigma D}, \frac{q}{\sqrt{gD^3}}, S\right] \tag{28}$$

In order to find the "critical flow number", F_{cr}, the depth was eliminated between the Weber number and the modified Froude number.

$$\begin{aligned}
F_{cr} &= f\left[q, S, T\right] = \left(\frac{N_W}{\sqrt{S}}\right)^{3/2}\left(\frac{N_F}{\sqrt{S}}\right)^{-1} \\
F_{cr} &= q^2\left(\frac{\rho}{\sigma}\right)^{3/2} g^{1/2}\, S^{-1/4} \\
F_{cr} &= \left(\frac{\rho q}{\sigma}\ \frac{V^{1/3}(gD)^{1/3}}{S^{1/6}}\right)^{3/2} \\
F_{cr} &= \left(\frac{\rho q V}{\sigma S^{1/6}}\right)^{3/2}
\end{aligned} \tag{29}$$

In order to find the critical depth number, N_D, the volume rate of flow was eliminated between the Reynolds number and the modified Froude number.

$$\begin{aligned}
N_D &= f\left(D, S, T\right) = R_D\left(\frac{N_F}{\sqrt{S}}\right)^{-1} \\
N_D &= D^{3/2}\,(gS)^{1/2}\,\nu^{-1}
\end{aligned} \tag{30}$$

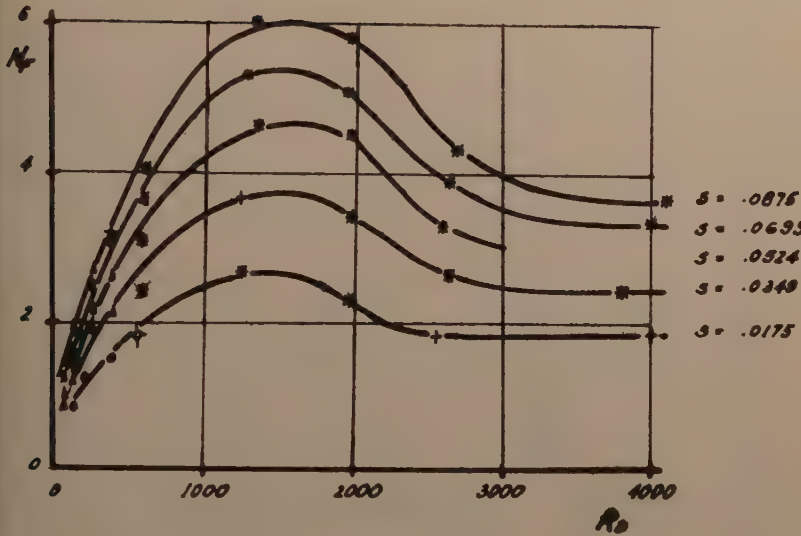


Fig. 12. Relation Between Froude and Reynolds Numbers at Various Slopes

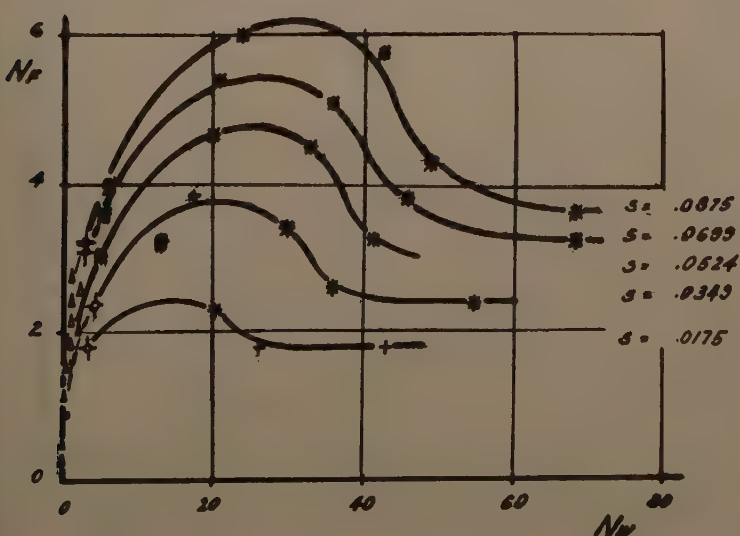


Fig. 13. Plot of Froude and Weber Numbers

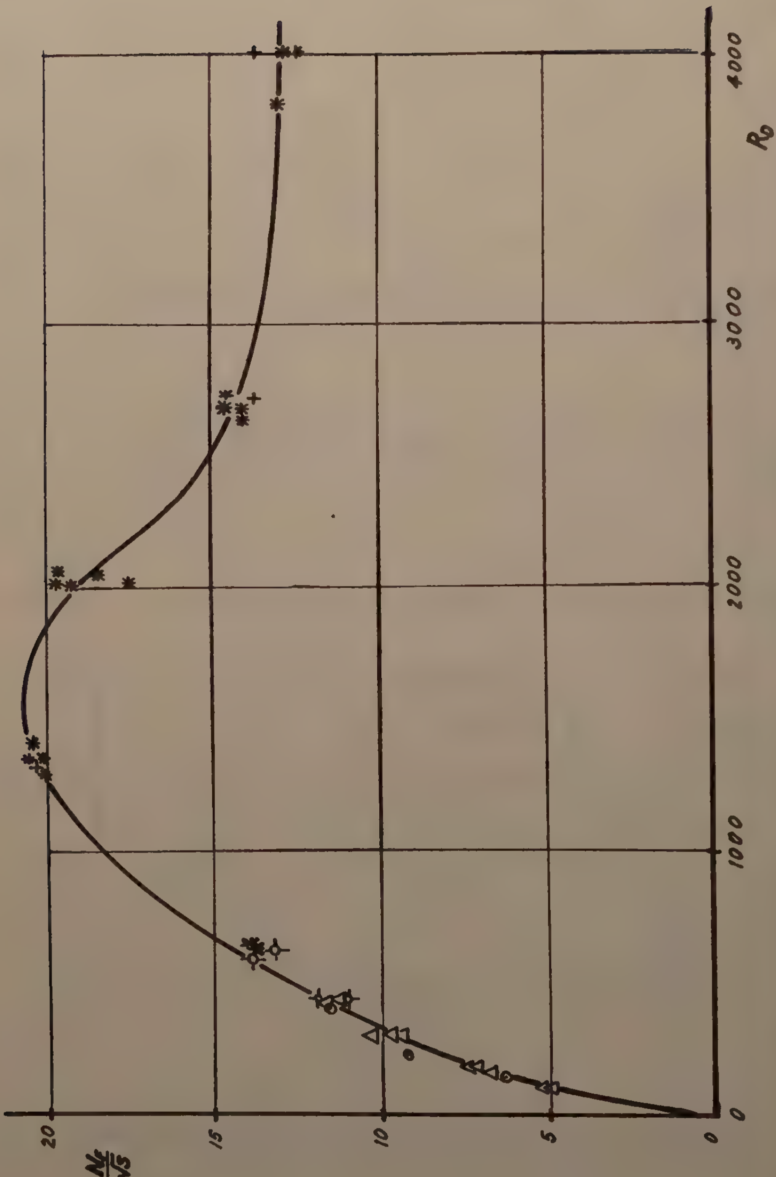


Fig. 14.—Plot of Modified Froude Number and Reynolds Number

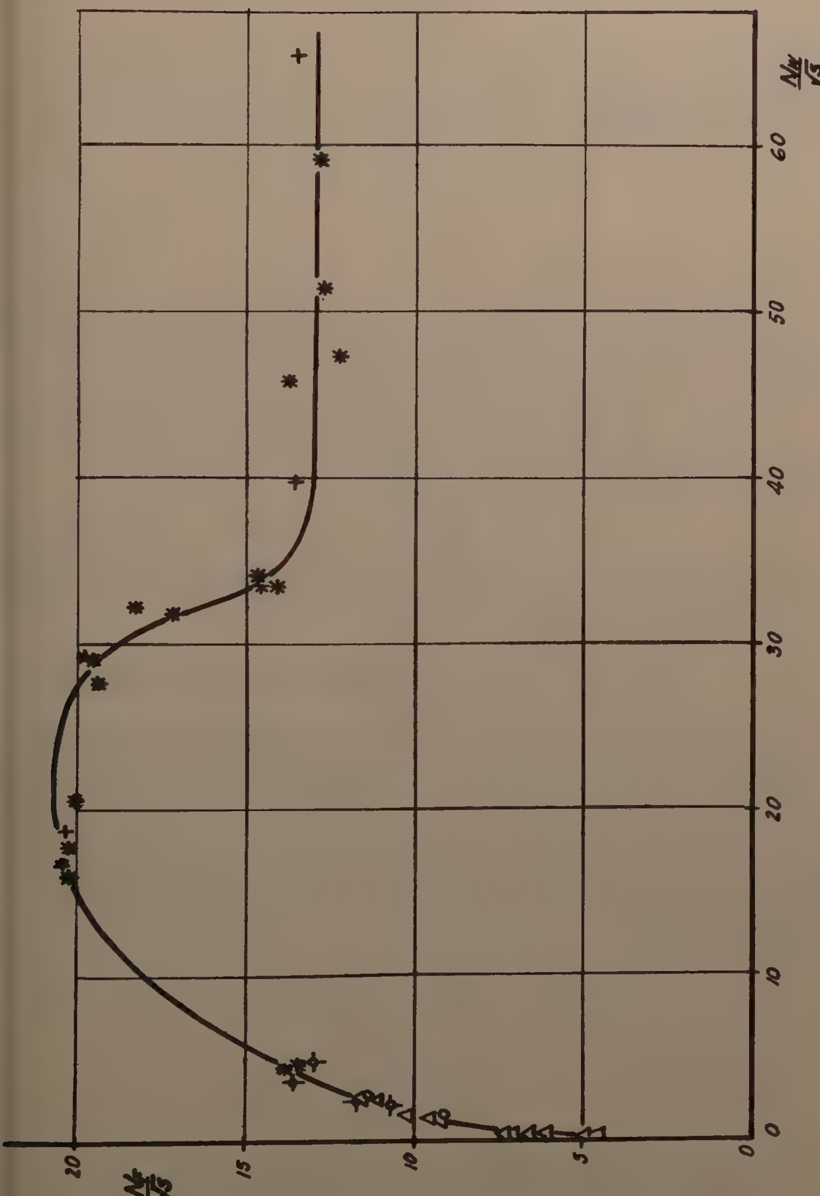


Fig. 15. Plot of Modified Froude and Weber Numbers

$$N_D = \left(g\frac{D^3 S}{\nu^2}\right)^{1/2} \qquad (31)$$

The plot of F_{cr} and N_D is presented in Fig. 16. In the region of instability which led to the formation of roll waves, the two parameters showed a straight line relationship. A mathematical formulation was possible below a $F_{cr} = 4$. The equation was

$$N_D = 28.1\ F_{cr}^{.218} \qquad (32)$$

The Meaning of F_{cr} and N_D

The "critical flow number" simply states that instability is imminent whenever the surface tension forces become comparable in magnitude to the momentum carried by the stream. This is equivalent to saying that the free surface energy becomes comparable to intrinsic energy of the stream.

The "critical depth number" is essentially a Reynolds number for laminar flow. The expression was developed earlier and given as Eq. (28). A Reynolds number may also be written as:

$$R_D = \frac{VD\rho}{\nu} = \frac{\rho V^2}{\mu V/D} \qquad (33)$$

which expresses the ratio of kinetic energy in the stream to the energy dissipated by viscous shear. Thus, the parameters, F_{cr} and N_D, have perfectly sound physical interpretations.

Experimental Relationship Between Reynolds Number, Froude Number, and Slope

Theoretical considerations predicted definite relationships of the above parameters when a parabolic velocity distribution was assumed. All experimental data were plotted in Fig. 17 to show the general inter-relationships. The instabilities which led to the formation of roll waves were in the region of supercritical laminar flow as predicted. Some instabilities leading to slug flows satisfied the predicted conditions, but most data indicated that slug flows were a transitional phenomenon and the actual velocity distribution may have been different.

Eqs. (17a) and (19) predicted that:

$$S_{crit} = \frac{12}{R_D}$$

$$N_F = \sqrt{5/3}\ \sqrt{R_D}$$

Roll waves were observed up to Reynolds numbers of 410, 413, 419, and 418. Then, for a mean $R_{D max} = 415$, the minimum slope for the formation of roll waves was, according to (17a):

$$S_{min} = .029 \sim 3\%$$

which was verified by the experimental results. The corresponding Froude number was experimentally equal to $N_F = 2.06$. At the higher Reynolds

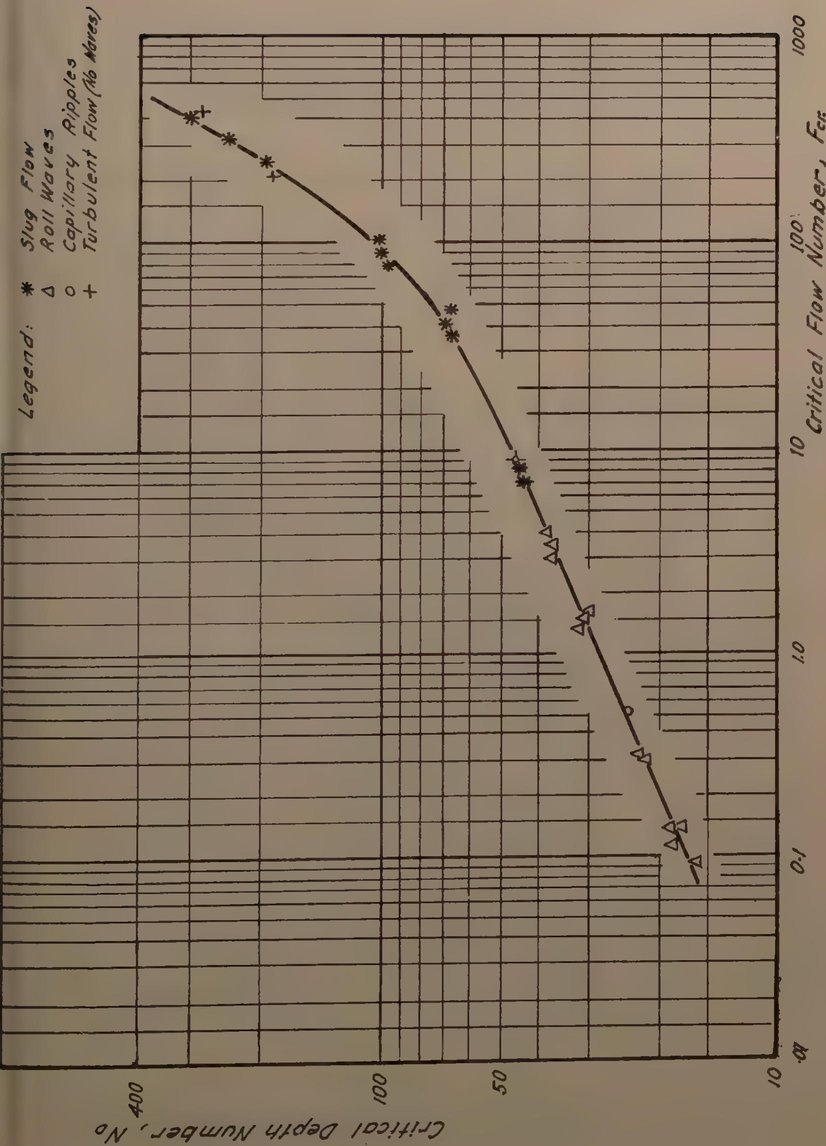


Fig. 16. Plot of Critical Depth and Critical Flow Numbers

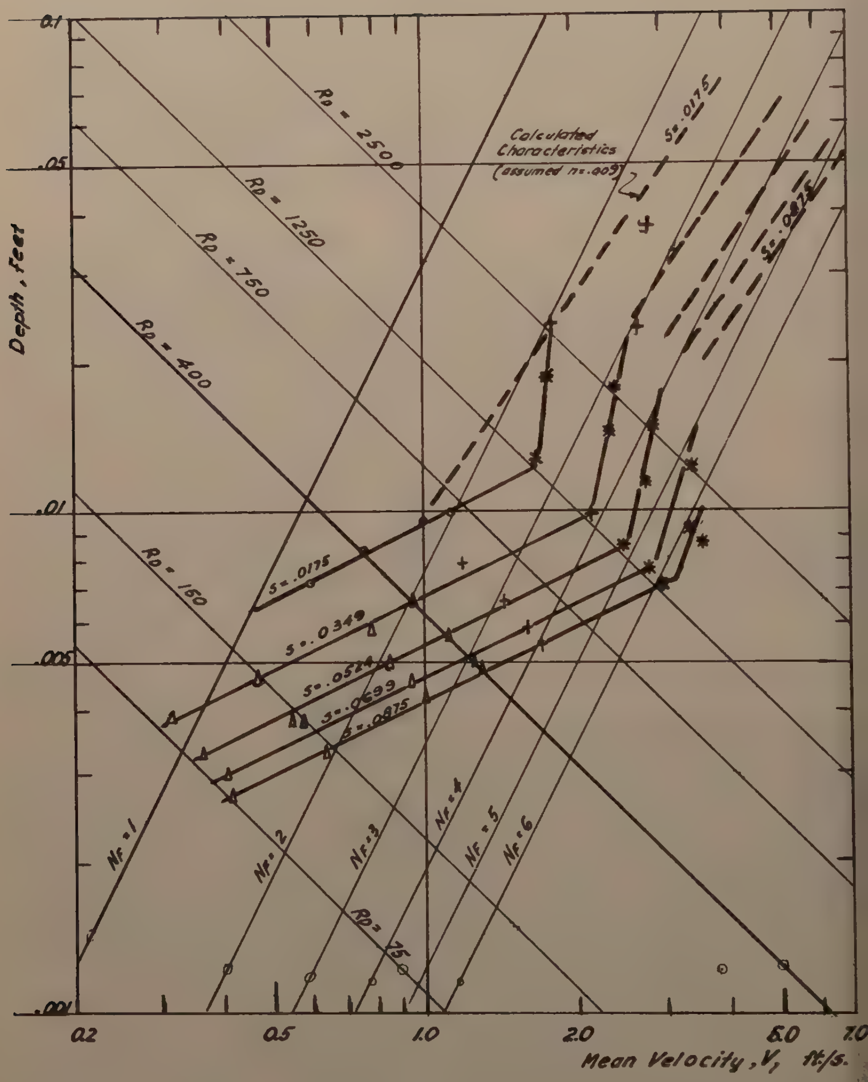


Fig. 17. Relations Between Depth and Velocity at Various Slopes for Flows in Laminar, Transitional and Turbulent Regions

umbers, the flow often degenerated into turbulent spots below the overflow entrance to the flume, before uniform flow conditions were obtained. Instabilities which led to the formation of slug flows were usually observed at Reynolds numbers larger than 1250. Turbulent spots were produced below this value by artificial rainfall. The deviation from the theoretical curve (Fig. 18) also starts at a Reynolds number of 1250. With a theoretical minimum Froude number of $N_F = 2$ and an approximate lower limit of Reynolds numbers of 200, the minimum channel slope for the formation of slug flows is:

$$S = .01$$

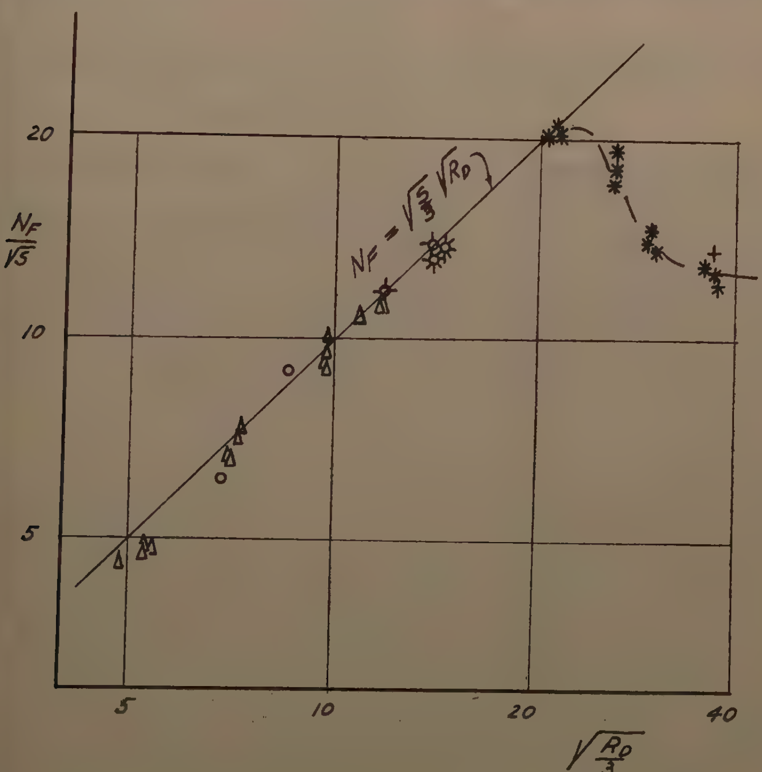
ctually slug flows were observed experimentally on a slope of

$$S = .0175 \sim 2\%$$

ll Froude numbers of flows leading to the formation of slug flows were above the theoretical minimum $N_F = 2$.

ocation of Initial Surface Instability

It was observed that the location of the initial disturbances leading to roll ave formation moved downstream with increasing Reynolds numbers. An



. 18. Relationship Between Froude Number, Reynolds Number and Slope

opposite trend was apparent with slug flows. A plot of the "length Reynolds number" and the "depth Reynolds number" elucidates this phenomenon (Fig. 19).

The plot does not reflect the locations of instability for artificially disturbed flows. Finite disturbances from the generator caused the instabilities to occur further upstream in the case of roll waves. For slug flows, the effect of the generator was less pronounced. Whenever the Froude number was in excess of the theoretical minimum of $N_F = 2$, turbulent spots could be brought about by raindrops onto the channel or by roughening the channel bed with sand grains.

A prediction of the location of the initial surface disturbances cannot be made accurately for channels with entrance conditions and channel linings different from those of the experimental flume.

The Formation of Roll Waves

The flow in the inclined open channel remained undisturbed on slopes up to approximately 1%. Surface disturbances in the form of ripple flow occurred on all tested slopes. Ripple flow was characterized by transverse ridges of short wave length. The speed of these waves was always larger than the surface velocity and equal to:

$$V_w = V + C_{min.}$$

where the minimum celerity, $c_{min.}$, under experimental conditions was:

$$C_{min} = .763 \text{ ft./sec.}$$

At conditions not sufficient to form roll waves, the ripple flow continued downstream over the entire length of the test flume.

Roll waves did not form on slopes below 3%. The formation of roll waves is shown schematically on Fig. 20. Perfectly smooth laminar sheet flow first suffered very slight undulations. A breakdown of the flow into ripple flow followed immediately. Then, a confused zone of rearrangement preceded the final emergence of roll waves. Plate I shows a photograph of this process. Roll waves were characterized by transverse ridges of high vorticity. The zones between the wave crests remained quiescent. (Plates IV, V.)

The Formation of Slug Flows

Slug flows originated with local disturbances. The agitated regions spread transversely and were swept downstream at the same time (Plate VI). A schematic representation of spot growth is shown in Fig. 21. A comparison was made between the experimental results of wind tunnel investigations and the development of the disturbed regions in the inclined open channel. The envelopes of spot growth behaved similarly for Schubauer and Klebanoff's[11] wind tunnel measurements and Amein's[12] result in the test flume (Fig. 22).

The regions between the spots were quiescent. The upstream front of the disturbed regions moved with a velocity approximately equal to 50% of the undisturbed surface velocity, the downstream front at about 80%. The upstream front resembled a traveling oblique hydraulic jump; the downstream end developed into a bore. Slug flows resulted when the quiescent pockets were eliminated by successive bores which telescoped to form the characteristic saw-tooth profile.(Plate VII.)

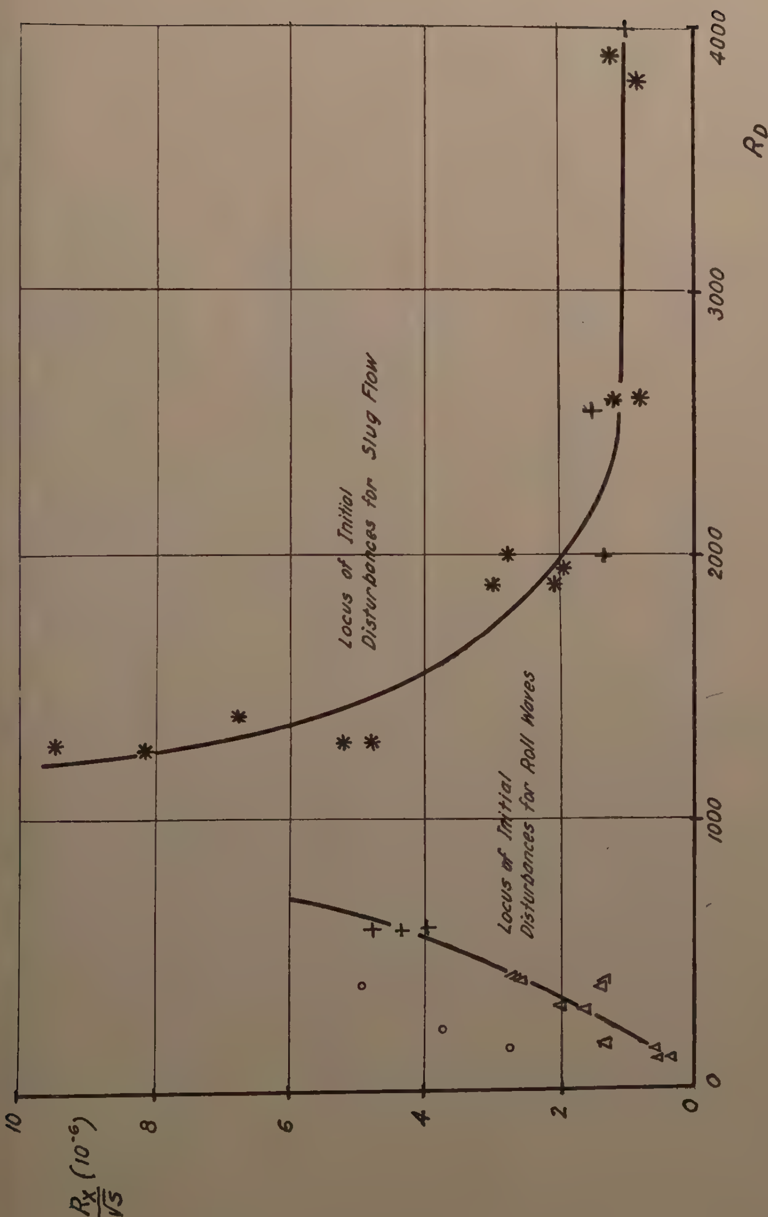
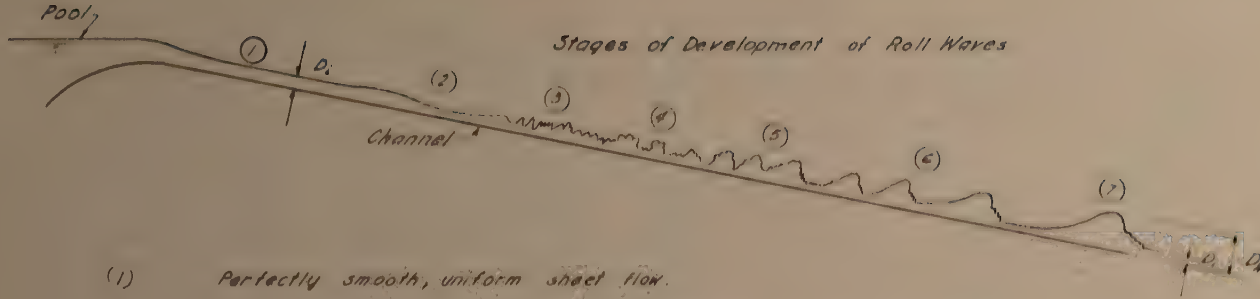


Fig. 19. Plot of Modified Length Reynolds Number and Depth Reynolds Numbers Showing Location of Initiation

Stages of Development of Roll Waves



(1) Perfectly smooth, uniform sheet flow.

(2) Smooth sinusoidal wave, perceptible by eye but not measurable.

(3) Transverse capillary ripples of larger curvature, shorter wave length with measurable height and frequency.

(4) Zone of decay and rearrangement, some waves coalesce and become dominant transverse ridges.

(5) Roll waves emerge as wave clusters steepen frontward and flatten behind the crest.

(6) Faster moving roll waves overtake smaller waves, thus growing in size. Also wave length increases, frequency decreases and boats quiesce.

(7) Roll waves continue to grow in height, speed and wave length. All vorticity is concentrated in crests, wave troughs become laminar. Some capillary ripples remain always at toe of roll wave.

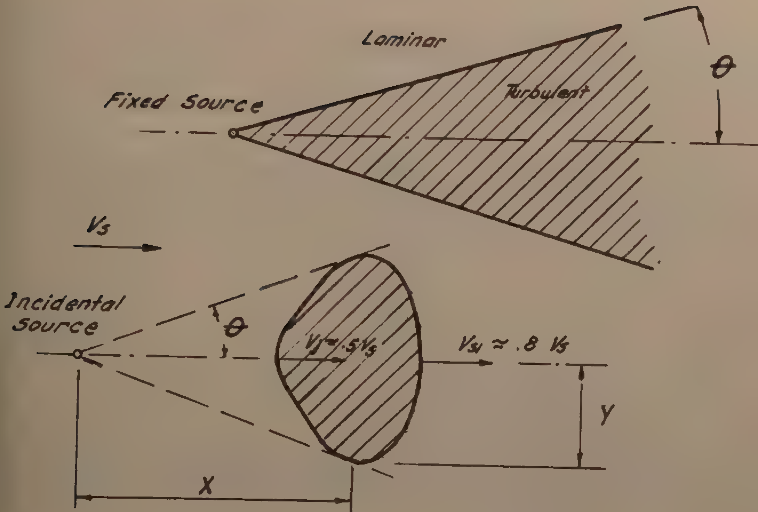


Fig. 21. Growth of Turbulent Spot

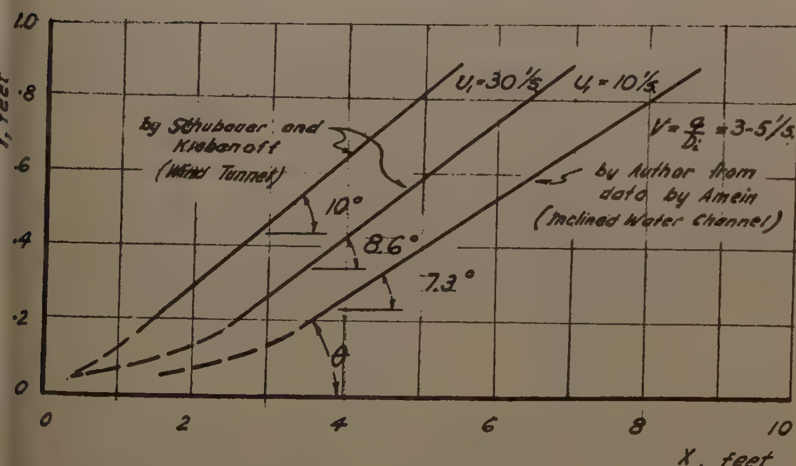


Fig. 22. Envelopes of Spot Growth

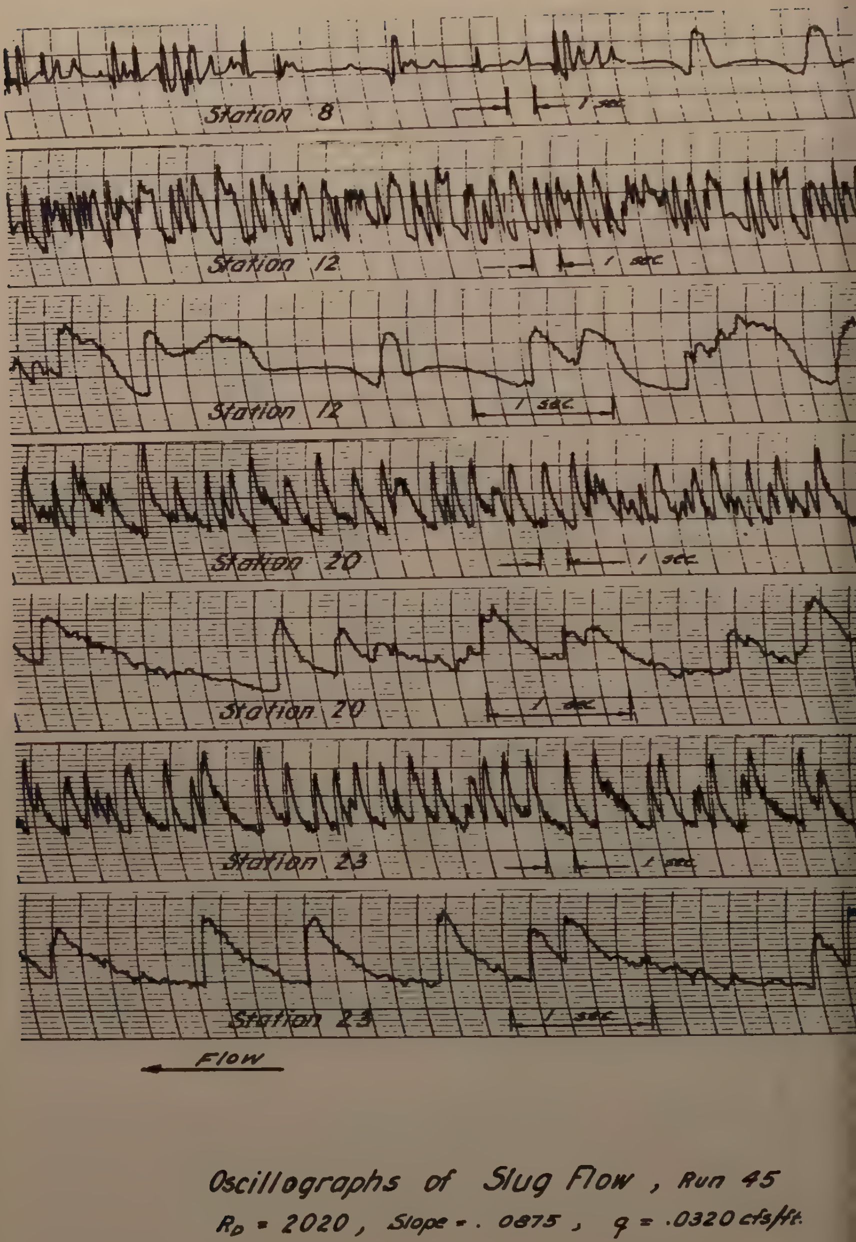


Plate VII. History of Slug Flows

Initially smooth water was pierced by occasional bursts of turbulence. The ot incidence increased as the flow progressed downstream. An increase in ot frequency was also observed with increasing Reynolds numbers. The sturbed regions grew as previously described. A drawdown of the water rface corresponded to a decrease in flow. This observation corresponded the "slackening" of the current reported by Cornish[13] half a century ago. late VIII.)

Slug flows were characterized by highly turbulent intermittent surges. ansverse ridges separated the regions of agitated flows and numerous naller waves were seen to ride the larger surges. During the formative riod, some waves were reflected from the side walls and could be seen as rved wave fronts. Eventually, the waves coalesced into a common transrse ridge.

ave Characteristics

For roll waves, the characteristics of wave length, wave height, and veloci- were investigated. A comparison of the data of wave height and wave ve- city showed that a terminal roll wave velocity existed. The relationship is otted in Fig. 23. The terminal velocity was then related to channel slope and ynolds number. Fig. 24 shows the data fitted to a curve and, on a log - log ot, the resulting equation was:

$$\text{Terminal } V_W = .44\, R_D^{1/3}\, S^{1/6} \qquad (34)$$

the same graph, the surface velocities were superimposed. It was seen t roll waves always travel faster than the undisturbed upstream surface ocity.

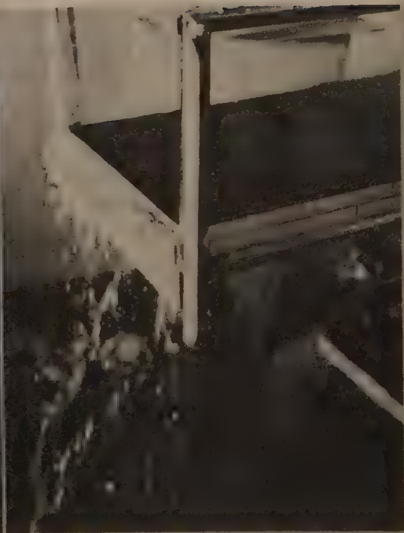
The roll wave characteristic described above was dimensional in nature. e orderlines suggested that a dimensional analysis might be profitable. In following analysis, both roll waves and slug flows were treated alike. rtinent flow and wave characteristics were grouped into the following di- nsionless parameters:

Reynolds number,	R_D
Froude number,	N_F
Dimensionless roll wave velocity,	$V_W +$
Dimensionless slug flow velocity,	$V_{sl.} +$
Depth ratios,	$D_{l.+}$, D_{h+}
Wave height - wave length ratio,	D_h/λ
Wave number,	V_W^2/fq

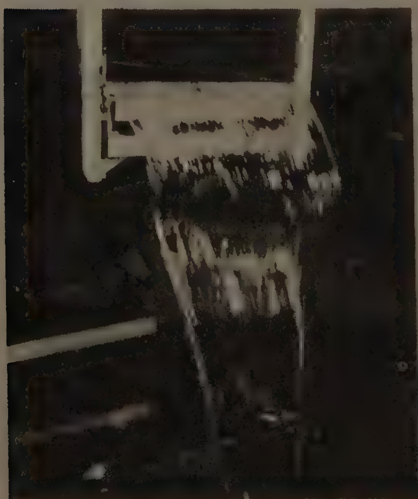
physical meaning of the wave number can be interpreted as "kinetic rgy per wave per unit discharge". In the presentation of the graphs, the al symbols were employed.

Fig. 24 already pointed out that the roll wave velocities exceeded the face velocities, and the terminal velocities were shown to be related to nolds number to the one-third power. A plot of dimensionless velocities Reynolds number showed the same trend for roll waves, but for slug s, the velocities were always less than the surface velocities, and, in

a) $R_D = 2650$, $s = .0699$, $q = .0427$ cfs/ft.



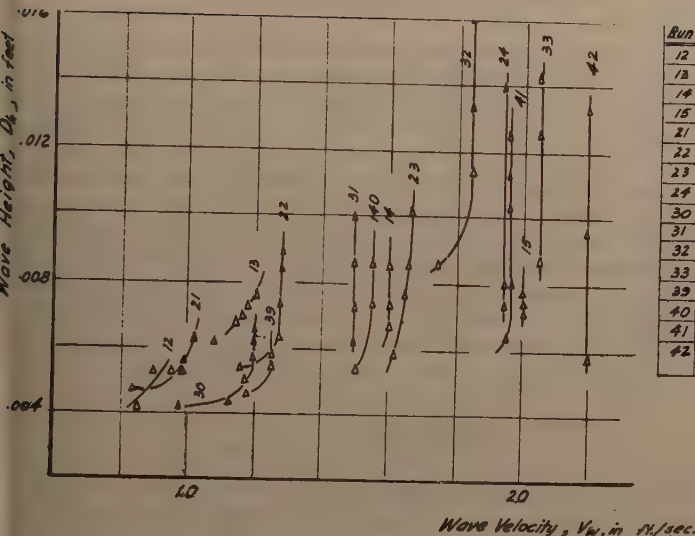
b) $R_D = 2650$, $s = .0699$, $q = .0427$ cfs/ft.



c) $R_D = 2600$, $s = .0524$, $q = .0427$ cfs/ft.

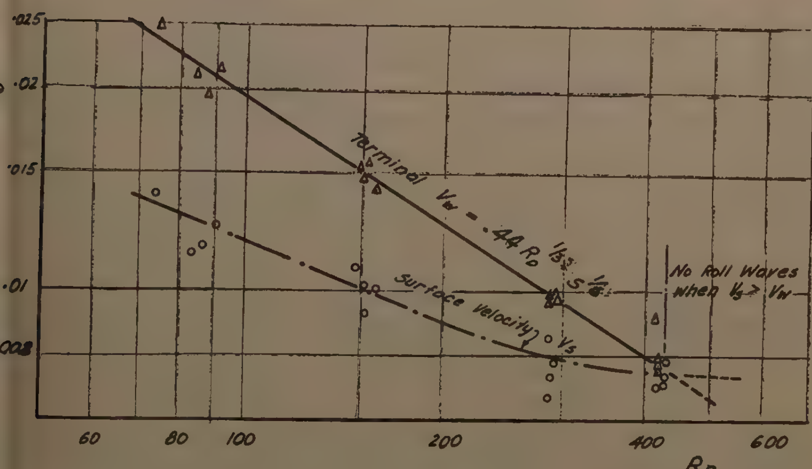
d) $R_D = 2600$, $s = .0524$, $q = .0427$ cfs/ft.

Plate VIII. Slug Flows Leaving Test Channel



Run	Slope	R_D
12	.0349	84
13	"	151
14	"	286
15	"	410
21	.0524	87
22	"	157
23	"	286
24	"	413
30	.0699	91
31	"	159
32	"	290
33	"	419
39	.0875	74
40	"	149
41	"	288
42	"	418

g. 23. Relationship between Wave Height and Wave Velocity for Roll Waves



g. 24. Terminal Roll Wave Velocity as a Function of Reynolds Number and Slope

in dimensionless form, showed a constant ratio independent of Reynolds number (Fig. 25).

When all the experimental data was plotted in terms of the modified wave number and the ratio of wave height to wave length, a narrow region delineat the zone of wave formation. The data pertaining to the steeper slopes plotted at the lower portion of the loop, so that the characteristics of roll waves and slug flows on still steeper slopes can be fairly well predicted (Fig. 26).

The plot involving the wave number pointed to the fact that this parameter tends to zero with increasing discharge. Slug flows were characteristic at larger discharges and plotted below wave numbers pertaining to roll waves. At very large discharges, the wave number vanishes. This reflects the fact that roll waves and slug flows are phenomena observed only at relatively low discharges. Although the continued growth of wave height and wave length made the length of the experimental channel seem insufficient, the ratio of wave height to wave length approached a limiting value (Fig. 26).

CONCLUSIONS

Steady flows in inclined open channels degenerated into periodic wave patterns under specific conditions of slope and discharge. The sources of

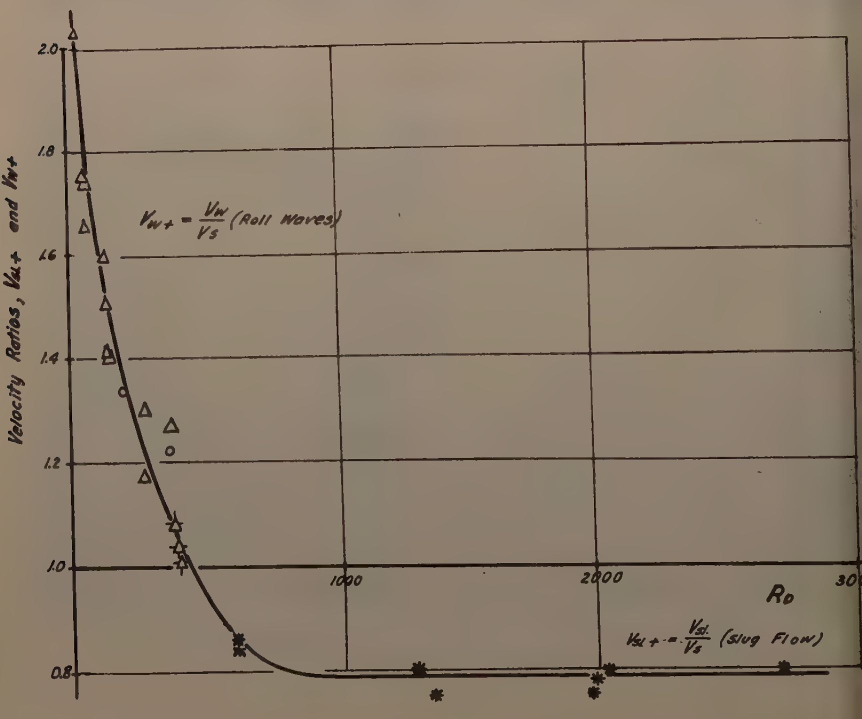


Fig. 25. Plot of Velocity Ratios and Reynolds Number at Station 23

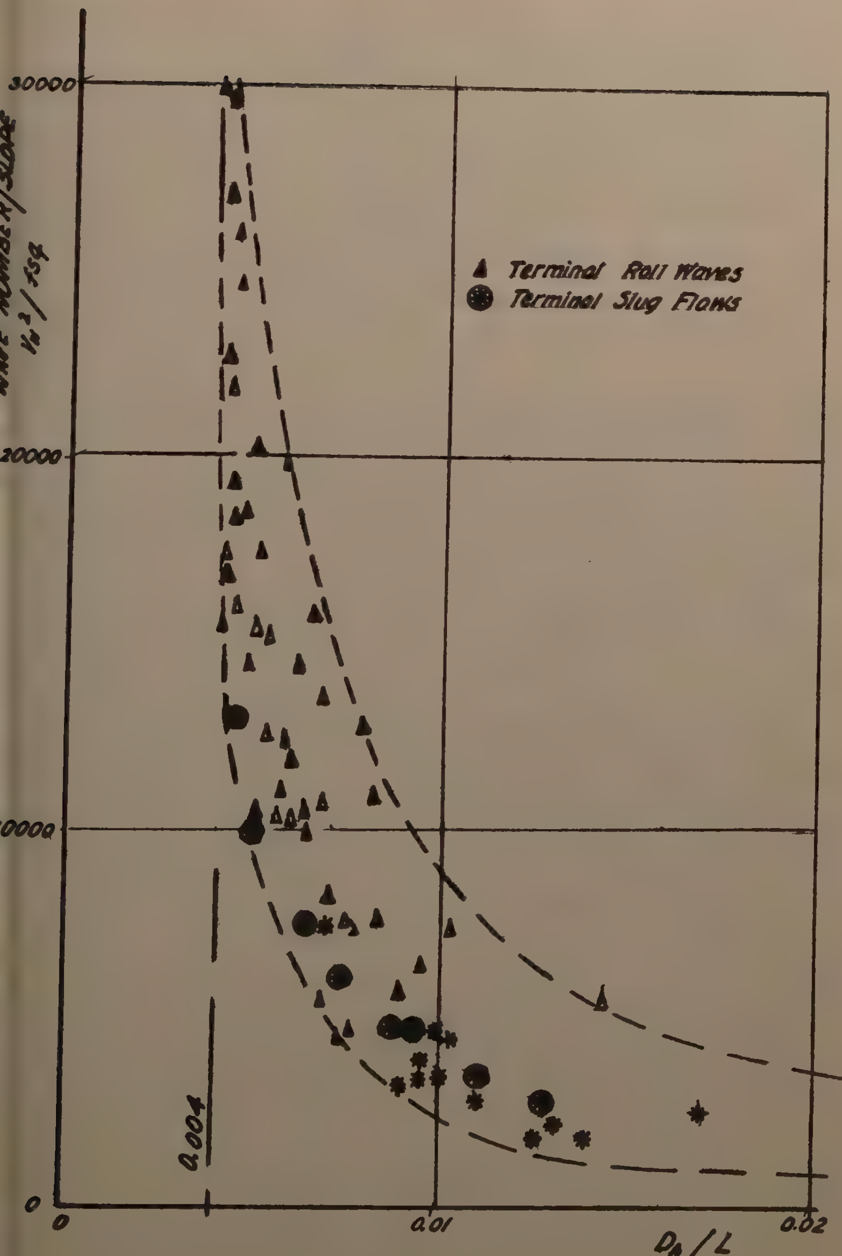


Fig. 26. Plot of Modified Wave Number and Ratio of Wave Height to Wave Length

instability were either carried by the stream or were externally superimpos perturbations. Two distinct types of wave trains were delineated. They wer designated as roll waves and slug flows respectively.

Roll waves formed whenever the surface velocity of the undisturbed strea was less than the minimum velocity of surface waves, provided that the channel slope was sufficiently steep. Experimentally and analytically, the minimum slope sufficient for roll wave formation was about 3%. The maximum Reynolds number of the undisturbed stream was 420. Roll waves resul ed from the interaction of surface tension and gravity forces at a slightly dis turbed surface. They were characterized by transverse ridges of high vorticity. The regions between the crests were quiescent. The terminal veloci ties of roll waves were in excess of the surface velocity of the undisturbed stream. The ratio of wave height to wave length approached the limiting val of 0.004.

Slug flows resulted from instabilities which caused the transition from supercritical laminar flow to turbulent flow. The initial disturbances were carried by the stream or resulted from interference by the side walls. The transition was a random phenomenon and could not be ascribed to any partic lar boundary condition. Locally disturbed regions spread transversely and contaminated adjacent zones. They were swept downstream like traveling oblique hydraulic jumps and seemed analogous to the turbulent spots of boundary layer studies in wind tunnels.

The phenomenon of the "traveling oblique hydraulic jumps" represented a discontinuity in open channel flow. In wind tunnel investigations, the turbule spots were not considered to be "shock" waves. Although one may consider shallow flow in an inclined open channel a boundary layer phenomenon, the additional force components of gravity and surface tension prohibit a quantitative analogy between compressible and incompressible fluid flows.

Slug flows formed whenever the surface velocity of the undisturbed flow was in excess of the minimum velocity of surface waves. They were charac terized by a succession of highly turbulent surges. The wave crests were separated by agitated regions. The terminal velocity of slug flows was less than the surface velocity of the undisturbed stream. In the test flume, the range of Reynolds numbers was from approximately 1200 to below 4000. At Reynolds numbers of 4000, the resulting flow was thoroughly turbulent and n slug flows were observed. The minimum slope for slug flow formation was approximately 2%. The Froude numbers were always in excess of 2. The flow decreased noticeably between waves and the surges literally jumped of the experimental flume. In prototype channels, slug flows are known to occu also at much higher Reynolds numbers.

During particular test runs, not all waves were of the same size or trave with the same speed. The deviations from the mean were at times considera ble and the concept of waves "that are alike in size and shape and travel wit a constant velocity" which was expressed by Thomas, Dressler, and other writers must be questioned. In this study, the analysis of the oscillographs well as the measurements of wave velocity reflect average values.

Roll waves and slug flows were analyzed and plotted in terms of pertinen dimensionless parameters. Flow conditions susceptible to instability were delineated by a "critical depth number" and a "critical flow number". Signi cant wave characteristics were expressed by a "wave number".

Roughened boundary conditions enhanced the formation of roll waves. R waves, the phenomenon often observed on fairly rough street surfaces, are

sentially roll waves, and it seems that they will form, regardless of the undary conditions. Experimentally, slug flows were also precipitated by undary roughnesses. However, the disturbance sources were isolated three-nensional obstructions (sand grains). A sufficiently large number of dis-bance sources, that is, a very rough channel, agitates the flow sufficiently it slug flows cannot form. This seems to be substantiated by the fact that ig flows are only observed in man-made channels and are not characteristic swift mountain streams. The roughening of the channel as a measure to minate slug flows was pointed out by H. Rouse.(14)

The theory of roll waves and slug flows presented in this study presupposed it the velocity distribution was significant. A parabolic distribution was sumed and was found quite satisfactory. Jeffreys(15) reported earlier that ig flows (alias roll waves) were the transition from "turbulent flow with a arly plane surface to turbulent flow marked by transverse ridges".

The pertinent characteristics of roll waves and slug flows were expressed terms of dimensionless parameters. In order to test their validity, a simi- study could be undertaken with a different fluid. A longer channel capable steeper slopes would also extend the range of this study.

Disturbances leading to the formation of roll waves could be superimposed on the laminar stream by controlled means. In the experiments, randomly ierated disturbances led to the same periodic wave formation with the same ected frequency that evolved when an undisturbed stream degenerated into l waves. The frequencies and amplitudes preferentially leading to roll ve formation could be studied by means of a vibrating ribbon or controlled ctric pulses. Similar methods have been employed successfully in wind nel studies.

Slug flows were reported to lead to air entrainment on dam spillways. Maistre and R. Oblensky(16) published photographs which suggested that entrainment was caused by surface roughness. Thus, it seems incongruous t rough channels eliminate slug flows but generate air entrainment, if air rainment is preceded by the former. A further study with a rough channel planned and could elucidate also the proper significance of the two phe-nena in respect to each other.

ACKNOWLEDGEMENTS

The author acknowledges gratefully the constructive criticism and kind en-ragement by the late Professor Andre L. Jorissen, M. ASCE. Professor lville Priest, M. ASCE, suggested the study. Useful suggestions and erences were contributed by Dr. L. Crabtree, Cambridge, England, while latter was at the Graduate School of Aeronautical Engineering of Cornell versity.

Notations and Definition of Terms

Depth of flowing water upstream from disturbance

Depth at point of apparent initiation of disturbance

Depth of average wave crest

Channel station at point of initiation of disturbance

q	Discharge per unit width	
V	Mean velocity	
V_s	Surface velocity	
V_w	Velocity of roll wave	
V_j	Velocity of turbulent spot (jump)	
V_{sl}	Velocity of slug (bore)	
V_{w+}	Velocity ratio V_w/V_s	
V_{sl+}	Velocity ratio V_{sl}/V_s	
ρ	Mass density of water	
σ	Surface tension	
ν	Kinematic viscosity	
f	Frequency of waves	
N_F	Froude number	$V/\sqrt{gD}$
R_D	Reynolds number in terms of depth	$\frac{VD}{\nu}$
R_X	Reynolds number in terms of station	$\frac{VXi}{\nu}$
N_W	Weber number	$\rho\frac{V^2D}{\sigma}$
S	Channel slope	
N_D	Critical depth number	$\left(g\frac{D^3S}{\nu^2}\right)^{1/2}$
F_{cr}	Critical flow number	$\left(\rho\frac{qV}{\sigma\, S^{1/6}}\right)^{3/2}$

All terms are expressed in basic units of the English system.

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Journal of the

HYDRAULICS DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION

e: Paper 2097 is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 85, HY 7, July, 1959.

EISENHOWER AND GRASS RIVER LOCK MODELS[a]

Closure by Martin E. Nelson and Harvey J. Johnson

MARTIN E. NELSON,[1] F. ASCE, and HARVEY J. JOHNSON,[2] M. ASCE.—Ir. Webster's discussion showing a good correlation between model and rototype hawser forces is a valuable addition to the field of experimental ock hydraulics. In view of the numerous factors involved in such a comparison, it is reasonable that a closer correlation could not readily be obtained rom a very limited number of tests.

The hawser forces given in the discussion cannot be compared directly vith those in the basic paper. In some instances the hawser force per ton of ow or ship displacement can be compared but in this instance the lifts are too lissimilar. Other factors influencing such a comparison, to name a few, are ocation of tow in the lock, rates of filling or emptying, depth of water cushion nder the vessels, and the elasticity of the restraining hawsers.

The senior author has observed the operation of the prototype Eisenhower nd Snell (formerly Grass River) locks. The water surfaces in the chamber nd lower approach were very smooth and the valves functioned quietly. In hese respects there was good agreement between the model and prototype. Although accurate tests have not been made in the prototype locks, it appears hat their hydraulic systems are somewhat more efficient than their respective nodels indicated, the time required to fill or empty the locks being somewhat horter. This type of discrepancy between model and prototype performance vhich has been observed in other locks and in outlet structures, is probably lue to lack of exact similitude in conduit roughness. The vortical activity at he intake manifolds was more pronounced in the full scale locks than in the nodels.

.. Proc. Paper 1582, April, 1958, by Martin E. Nelson and Harvey J. Johnson.

.. Hydr. Engr., St. Paul Dist. Corps of Engrs., U. S. Dept. of the Army, St. Paul, Minn.

:. Hydr. Engr., U. S. Dept. of Agriculture, Agriculture Research Service, Soil and Water Conservation Research Div., St. Anthony Falls Hydr. Lab., Minneapolis, Minn., formerly with the Corps of Engrs., U. S. Dept. of the Army, St. Paul, Minn.

RADAR FOR RAINFALL MEASUREMENTS AND STORM TRACKING[a]

Discussion by J. R. Bowman

J. R. BOWMAN,[1] M. ASCE.—The author is to be commended for his omprehensive progress report on the status of the use of radar as a means or making quantitative precipitation forecasts. He also should be thanked for is sparing use of the involved terminology that has become the language of he radar meteorologist. Although radar meteorology is still an infant in ears, a tremendous amount of research has been undertaken in this field; the engthy bibliography appended to this paper merely highlights some of the nore significant developments in the subject field of endeavor. Owing to the ature of its original presentation, the paper is necessarily brief, and many orthwhile points suffer from lack of clarification.

In conjunction with hobby pursuits in the electronics and meteorological ields, combined to some extent in the form of radar meteorology, the writer as come to recognize many of the potential uses, as well as the limitations, f radar as a meteorological instrument. The greatest deterrent to the outight purchase of radar equipment that has been designed specifically for neteorological use probably is its first cost. By way of illustration, the cost f each high-power, 10-centimeter, weather radar unit currently being placed n service by the United States Weather Bureau,[17] amounts to about 100,000, exclusive of the cost of associated structures and installation. In iew of such high costs, most of the weather radar units currently used by nstitutions other than Federal governmental agencies, are modified overnment-surplus units that were designed originally for military duty, but hich can be purchased for a fraction of their original cost. In military appliations, precipitation echoes are not desirable because they tend to mask or bscure the echoes received from the primary targets; for this reason, the ircuitry in military radar units is designed to enhance the echoes from the esired targets and to suppress precipitation echoes. Modification of a miliry radar unit is largely confined to the alteration of the video presentation ircuits in the receiver section, inasmuch as any significant change in basic eceiver or transmitter circuits could involve the redesign of the entire radar nit. Thus the advantage of low first cost of surplus radar equipment (the uthor's "discarded equipment") frequently is offset by compromises in perrmance.

The author's complaint concerning "equipment of improper wavelength" is stifiable, particularly where it concerns 3-cm radar applied to quantitative ork. It has been determined experimentally that for showers of 16 mm/hr tensity,[18] the signal attenuation at 3 cm is roughly 60 times greater than

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Proc. Paper 1901, January, 1959, by Glenn E. Stout.

Civ. Engr., Erik Floor & Assocs., Chicago, Ill.

at 10 cm. At greater intensities this difference is even more pronounced. Numerous studies of field situations of various types lend support to the growing belief that 10-cm radar is more versatile and more stable than either the 3-cm or 5-cm radars. The decision of the U. S. Weather Bureau to order 10-cm radars further strengthens this belief.

The major deterrent to the more extensive use of 10-cm units again is the first cost. Dimensions of the larger components of a radar unit are about proportional to the wavelength, so that the volume occupied by a complete 10-cm unit may be several times greater than the space occupied by a 3-cm unit of comparable power rating.

From the foregoing discussion, it might appear that 3-centimeter radar has little value as a meteorological instrument; this is not the case. Although the 3-cm band may not be the optimum band for quantitative prediction of precipitation at long range, it has considerable utility in determining the areal extent and vertical development of potentially severe storms. This has been demonstrated conclusively by the author and his associates in Illinois, and through its adoption as a key unit in the tornado tracking networks of the Southwest. Some of the 3-cm units originally designed for military airborne use, such as the AN/APS-4, are so compact that they are readily converted to small mobile units and easily moved into relatively inaccessible areas; such a unit has been put to effective use by Decker(19) and others in the study of orographic precipitation in the Coast Range in Oregon.

The paper illustrates several of the methods by which correlation of radar echoes with observed precipitation has been attempted; all illustrate the need for further research. The writer questions the appropriateness of the comparisons drawn in Fig. 3. Hourly rainfall depths are compared with isoecho contours observed over a short period during each hour. This would imply that the rainfall intensities, as well as the distribution of heavy precipitation centers, as indicated by radar echoes, were assumed to remain constant throughout each hourly period. On the contrary, rainfall is notoriously variable in practically all types of storms. The writer suggests that an integrated series of time-lapse photographs, similar to those associated with the studies of mesosystems undertaken by the University of Chicago,(20) might represent more closely the ever-changing storm patterns.

The development of the pulse integrator (illustrated by Fig. 4) represents a giant step toward accurate correlation of echo and rainfall intensities. Its principal disadvantage, however, is that the time required for gating and integrating limits its effectiveness to relatively small areas.

The nature of the received power and rainfall intensity traces of Fig. 5 show considerable promise for accurate correlation. The errors inherent in comparisons of instantaneous values appear to be of a compensating nature, which suggests the possible improvement of accuracy by a time-integration method. Values of signal intensity could be converted to represent db values above a dbm cutoff value. In an approximate analysis of the left-hand group of curves, the writer obtained very close correlation by shifting the time-integrated rainfall curve 2 minutes to the left, and by integrating the signal intensity curve above a cutoff value of 65 dbm. The writer is not at all certain as to the significance of the cutoff value, but he ponders the possibility that it may be related to signal attenuation.

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PROBLEMS CONCERNING USE OF LOW HEAD RADIAL GATES[a]

Discussion by E. Pariset and B. Michel

E. PARISET,[1] M. ASCE and B. MICHEL.[2]—Mr. Rhone's very interesting aper has brought up a number of questions concerning the use of low head adial gates.

Among the largest radial gates built is the 148 ft wide by 30 ft high gate uilt for Donzere Mondragon (France).

We had the opportunity to test, on scale models, several spillways using adial gates, so we are happy to add a few "stones" to the "edifice" built by Ir. Rhone.

About discharge characteristics, we generally use the fourth method: $= C\ D\ L\sqrt{2g\ H}$, as any formula is difficult to use for very large gate openigs when the discharge depends more of the crest profile than of the gate pening, and as for low openings, true orifice flow occurs.

The value of C depends not only of θ (angle formed by the tangent to the ate lip and the tangent to the crest curve at the nearest point of the crest urve), but also of the crest shape which governs to a certain extent the ressure on the spillway at the gate outlet. The thinner the crest profile by omparison with the "datum shape" the more likely some depression is to occur and the higher will be the coefficient of discharge, as shown on graph o. 1 giving the coefficient of discharge obtained for Edgard de Souza dam Brazil). At full gate opening the coefficient of discharge, using: $= C\ L\ H^{3/2}$, was C = 4.17.

This profile was tested and used for the surelevation of the dam.

A profile much thinner than the "datum shape", graph No. 1, was used not ly to obtain a higher discharge and to save on the volume of concrete, but ainly to allow the new profile of the surelevated dam to be tangent with the d one, in order to avoid the water nappe from jumping free of the downream face of the dam and falling out of the existing bucket at the foot of the m.

It must be noted that C varies also with the value of H, a greater head ineasing the depression and the coefficient of discharge.

All our tests have confirmed that if the gate is located just on the crest, e pressure downstream will be lower at low gate openings than at full gate enings, for the very reasons explained on page 47, of Mr. Rhone's paper.

Usually even a light shifting of the gate seal on the downstream side of the est is sufficient to keep the pressure for all gate openings higher or equal the minimum pressure occurring at full gate openings.

Proc. Paper 1935, February, 1959, by Thomas J. Rhone.
Director, LaSalle Hydr. Lab., Montreal, Canada.
Research Engr., LaSalle Hydr. Lab., Montreal, Canada.

DISCHARGE UNDER A RADIAL GATE

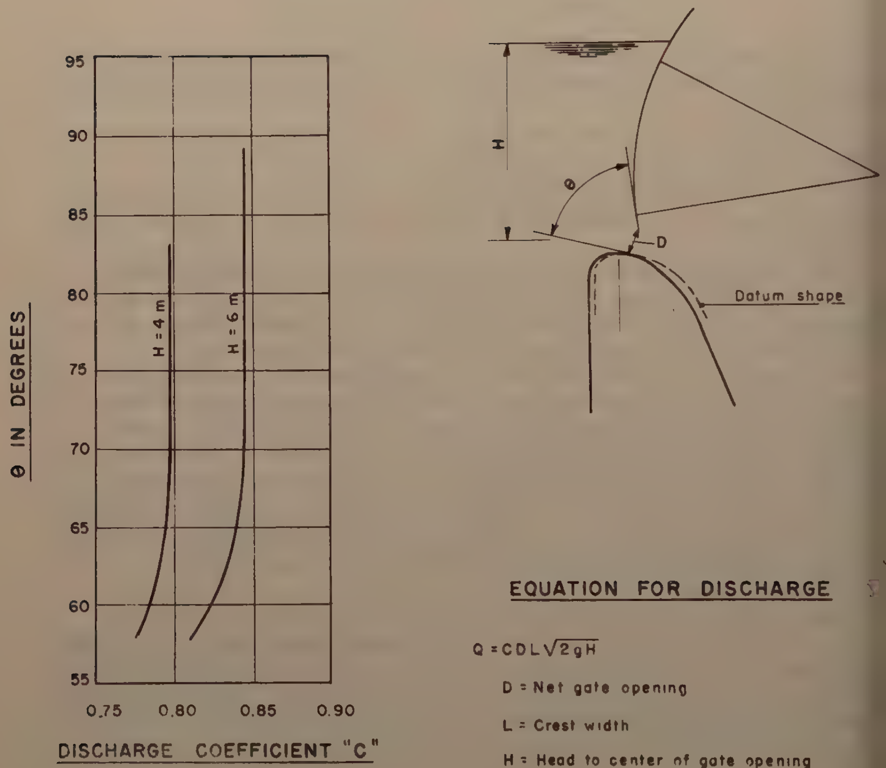

Fig. 1

Graph No. 2 shows very clearly the reduction of pressure obtained by moving downstream the gate seal of Edgard de Souza Dam.

The problem is exactly the same if a vertical lift gate is used instead of a radial one. We had the opportunity to test such an arrangement for the Hart-Jaune Spillway (Canada).

To determine the maximum allowable pressure on the profile of a spillway, we use a cavitation coefficient similar to the one used for hydraulic turbines.

$$\sigma = \frac{p/w + h_a - h_v}{V^2/2g}$$

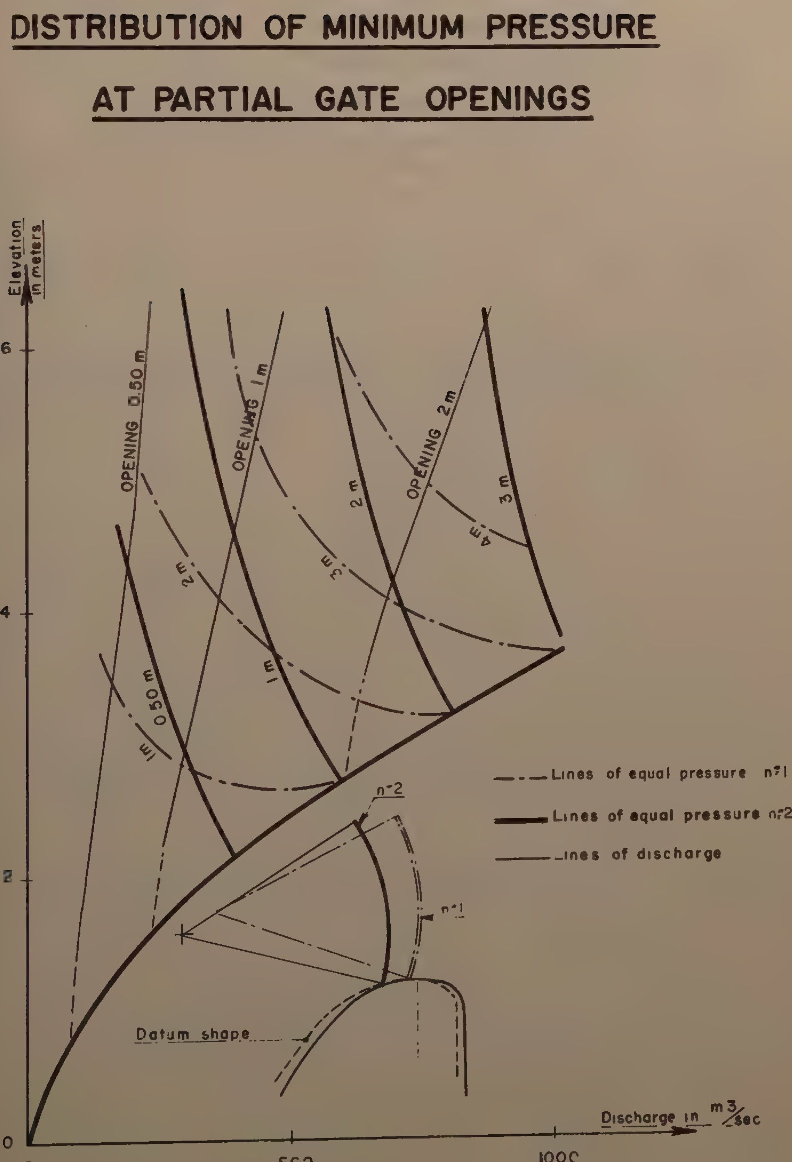
DISTRIBUTION OF MINIMUM PRESSURE
AT PARTIAL GATE OPENINGS
Elevation in meters
6
4
2
0
OPENING 0.50 m
OPENING 1 m
OPENING 2 m
0.50 m
1 m
2 m
3 m
4 m
Lines of equal pressure n°1
Lines of equal pressure n°2
Lines of discharge
n°2
n°1
Datum shape
Discharge in m3/sec
0
1000

Fig. 2

where: p/w minimum pressure head on the spillway surface near the gate

h_a atmospheric pressure head

h_v water vapor pressure head

$V^2/2g$ velocity head at the point of minimum pressure

The cavitation factor has the advantage of stating not only the sub-atmospheric pressure that could induce cavitation, but also the velocity head that could be transformed locally into additional negative pressure behind an irregularity of the surface.

For a spillway surface, the following meaning can be given to the cavitation factor(1)

$\sigma = 0$ certain cavitation

$\sigma = 0.5$ cavitation danger exists, but can be tolerated if spillway face is constructed very smoothly so that no irregularity will be present and if the spillway will operate under conditions giving these low σ values only a small percentage of the time.

$\sigma = 1.0$ no danger of cavitation

REFERENCE

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RELATIONSHIPS BETWEEN PIPE RESISTANCE FORMULAS[a]

Discussion by Peter Ackers

PETER ACKERS.[1]—The author's historical review of pipe resistance equations was paralleled by the writer in an almost simultaneous publication(1) in the United Kingdom, which drew the similar conclusion that the Colebrook-White(2) equation is the best means of estimating friction loss available at present. Unfortunately its complexity has prevented this equation coming into general use, there being two main obstacles to its adoption in engineering design:

(a) It cannot be expressed as a grid of straight lines on a simple design chart.
(b) If e, ν, R and S are quoted, V may be calculated directly, but in the common design case of known discharge and available slope, a trial and error process was needed.

The Moody(3) and Rouse(4) non-dimensional plots of the equation, though clarifying the picture, were only a partial solution of the problem facing the engineer. Neither Lamont's(5) addition of nomograms nor Bonnington's(6) transposition of the variables succeeded in reducing the application to the simple form required by the hydraulic engineer, i.e., design charts expressed directly in terms of conduit size, hydraulic gradient, velocity and discharge. However, studies of the problem at the Hydraulics Research Station have provided the required solution,(1,7) which is applicable to open channels as well as pipes.

The physical non-dimensional parameters, N_R, f and e/D are replaced by non-dimensional engineering parameters which separate the principle variables V, D and S, as follows:

$$\mathbf{V} = Vk/\nu \tag{1}$$

$$\mathbf{R} = \tfrac{1}{4}D/e \tag{2}$$

$$\mathbf{S} = 2gSe^3/\nu^2 \tag{3}$$

The Colebrook-White equation

$$V = -2\sqrt{(2gDS)}\,\log\left\{\frac{e}{3.7D} + \frac{2.51\,\nu}{D\sqrt{(2gDS)}}\right\} \tag{4}$$

becomes, in terms of these new parameters,

$$\mathbf{V} = -4\sqrt{(\mathbf{RS})}\,\log\left\{\frac{1}{14.8\mathbf{R}} + \frac{0.314}{\mathbf{R}\sqrt{(\mathbf{RS})}}\right\} \tag{5}$$

a. Proc. Paper 1962, March, 1959, by W. L. Moore.
1. Principal Scientific Officer, Hydrs. Research Station, Wallingford, Berkshire, U. K.

The advantages of this form of the turbulent-transitional resistance equation are:

(i) Being non-dimensional, it retains some physical significance and applies to any system of units.

(ii) The engineering variables, in parametric form, occur explicitly, so that for a given class of pipe and fluid,

$$V \propto \mathbf{V}$$

$$R \propto \mathbf{R}$$

$$S \propto \mathbf{S}$$

(iii) A resistance diagram applicable to any fluid or roughness can be plotted on the basis of these dimensionless engineering parameters, Fig. 1.

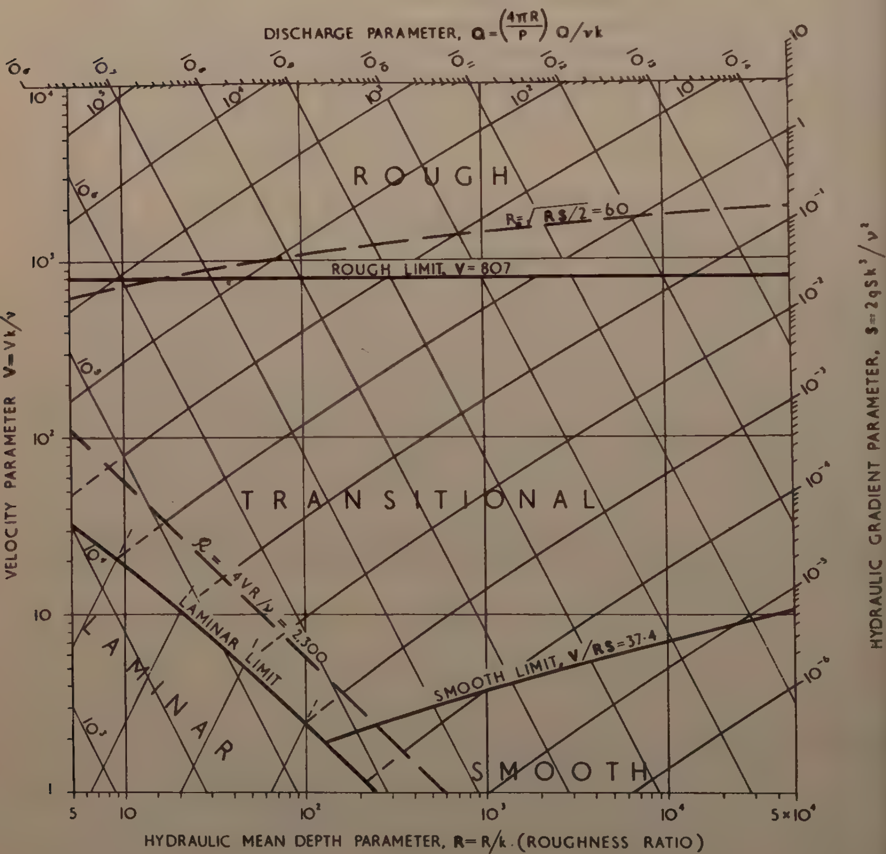

FIG. 1 THE USE OF THE COLEBROOK–WHITE FLOW FORMULA IN ENGINEERING DESIGN Universal Resistance Diagram.

(iv) For any given surface roughness and kinematic viscosity, this non-dimensional chart can readily be converted to a design chart in terms of V, R and S, merely by tracing the appropriate section and re-labelling the axes (example in Fig. 2).

(v) The addition of a grid of discharge lines provides a complete direct solution to flow problems in which any two of the four basic variables, V, R, S and Q are quoted.

There now seems no excuse for replacing the accepted resistance equation by exponential approximations, except in certain limited fields of hydraulic computation, such as the pipe-network example cited by the author. However, in ref. 7, the writer demonstrated how this can be done by partial differentiation of Eq. (5). Expressing the general exponential equivalent as:

$$V = C' R^{\alpha} S^{\beta} \tag{6}$$

it can then be shown that:

$$\alpha - \beta = 1.74 \sqrt{(2gRS)}/V \tag{7}$$

and

$$\frac{\alpha + 1 - 3\beta}{\beta - \frac{1}{2}} = \frac{e\sqrt{(2gRS)}}{2.32\nu} \tag{8}$$

whilst C' is given by simultaneous solution of Eqs. (6) and (4) for the chosen central values of R (or D) and S.

Alternatively, the relationship between α and β may be written

$$\frac{\alpha - \frac{1}{2}}{\beta - \frac{1}{2}} = 0.609\, R_* + 3 \tag{9}$$

where R_* is the roughness number, $e\sqrt{(gRS)}/\nu$. It follows from Eq. (8) that the oft-quoted criterion for dimensional homogeneity,

$$\alpha + 1 - 3\beta = 0 \tag{10}$$

is true only when e is zero, i.e., it is applicable only to smooth-turbulent conditions. This also results from Eq. (9) by putting R_* equal to zero, whilst with R_* large (rough-turbulence) β approaches 0.5.

The assumption implicit in the author's method of deducing exponential approximations from the Moody diagram is that relative roughness is constant. However, for any given class of commercial pipe, it is the absolute roughness which should be regarded as constant, and this is an important distinction if an equation covering various pipe diameters is required. To this extent, the writer believes his method to be more general than the author's.

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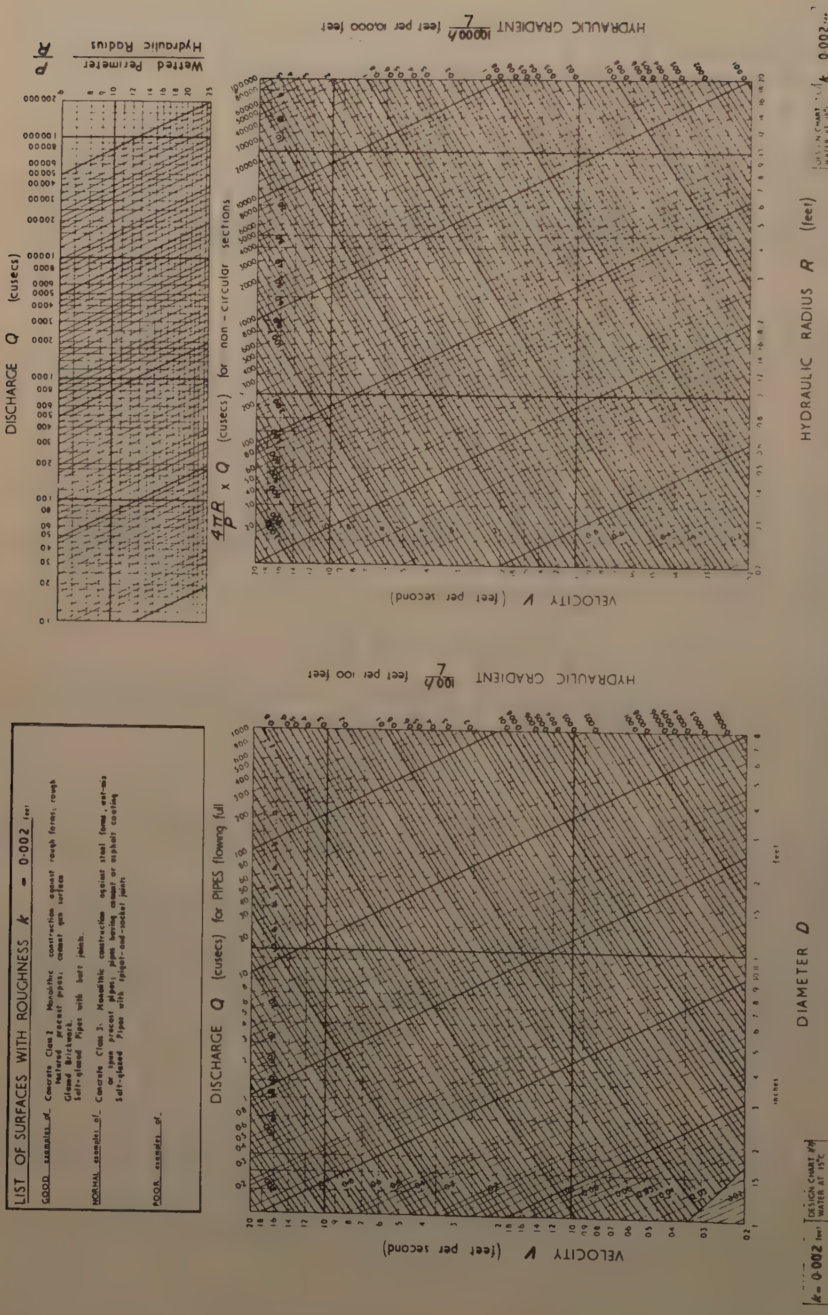

FIG. 2 EXAMPLES OF DESIGN CHARTS FOR PIPES AND CHANNELS

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PROCEEDINGS PAPERS

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RUARY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), 941(ST2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(HY2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), 950(HY2)[c], 1951(SM1)[c], 1952(ST2)[c], 1953(PO1)[c], 1954(CO1), 1955(CO1), 1956(CO), 1957(CO1), 1958(CO1), 959(CO1).

CH: 1960(HY3), 1961(HY3), 1962(HY3), 1963(IR1), 1964(IR1), 1965(IR1), 1966(IR1), 1967(SA2), 1968(SA2), 969(ST3), 1970(ST3), 1971(ST3), 1972(ST3), 1973(ST3), 1974(ST3), 1975(ST3), 1976(WW1), 1977(WW1), 978(WW1), 1979(WW1), 1980(WW1), 1981(WW1), 1982(WW1), 1983(WW1), 1984(SA2), 1985(SA2)[c], 1986 R1)[c], 1987(WW1)[c], 1988(ST3)[c], 1989(HY3)[c].

L: 1990(EM2), 1991(EM2), 1992(EM2), 1993(HW2), 1994(HY4), 1995(HY4), 1996(HY4), 1997(HY4), 1998 M2), 1999(SM2), 2000(SM2), 2001(SM2), 2002(ST4), 2003(ST4), 2004(ST4), 2005(ST4), 2006(PO2), 2007 W2)[c], 2008(EM2)[c], 2009(ST4)[c], 2010(SM2)[c], 2011(SM2)[c], 2012(HY4)[c], 2013(PO2)[c].

2014(AT2), 2015(AT2), 2016(AT2), 2017(HY5), 2018(HY5), 2019(HY5), 2020(HY5), 2021(HY5), 2022(HY5), 23(PL2), 2024(PL2), 2025(PL2), 2026(PP1), 2027(PP1), 2028(PP1), 2029(PP1), 2030(SA3), 2031(SA3), 32(SA3), 2033(SA3), 2034(ST5), 2035(ST5), 2036(ST5), 2037(ST5), 2038(PL2), 2039(PL2), 2040(AT2)[c]. 41(PL2)[c], 2042(PP1)[c], 2043(ST5)[c], 2044(SA3)[c], 2045(HY5)[c], 2046(PP1), 2047(PP1).

: 2048(CP1), 2049(CP1), 2050(CP1), 2051(CP1), 2052(CP1), 2053(CP1), 2054(CP1), 2055(CP1), 2056 Y6), 2057(HY6), 2058(HY6), 2059(IR2), 2060(IR2), 2061(PO3), 2062(SM3), 2063(SM3), 2064(SM3), 2065 T6), 2066(WW2), 2067(WW2), 2068(WW2), 2069(WW2), 2070(WW2), 2071(WW2), 2072(CP1)[c], 2073(IR2)[c], 74(PO3)[c], 2075(ST6)[c], 2076(HY6)[c], 2077(SM3)[c], 2078(WW2)[c].

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